Protection of Shanghai from Flooding

‘Open or closable navigational section?’

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PREFACE

This master thesis report signifies the end of my Master study at the Faculty of Civil Engineering and Geosciences, TU Delft. This project is realized with cooperation of ARCADIS Nederland B.V. Company.

It was my personal objective to investigate the flooding problem in Shanghai, which is quite close to my home city. Many thanks goes out to the graduation-committee who helped me to perform this research, Prof. Dr. Bas Jonkman, Dr. ir. H.G. Voortman, Ir. W.F. Molenaar and Ir. H.J. Verhagen.

I would like to thank Prof. Dr. Bas Jonkman, by providing a lot of freedom in my research in combination with his constant demand for help; he has contributed importantly to this study. Thanks to Ir. W.F. Molenaar for nine months' weekly updating meeting and providing inspiration and energy. Also many thanks goes to Dr. ir. H.G. Voortman, for making it possible to work in a Dutch company and supporting my 'crazy ideas'. Thanks to Ir. H.J. Verhagen for providing technical support at the final stage of this report. Further, I would like to thank my colleague Henry Tuin from ARCADIS, for fighting himself through a number of drafts of this thesis report.

Finally, special thanks is passed to Mrs. Ai Hong, the greatest woman who raised me alone in the last twenty years. Also special thanks to Sien and Junyan, for accompanying me through all the happy and difficult times in the Netherlands.

I hope the reading of this report is as pleasant as it was for me when I was working on it.

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ABSTRACT

Due to Shanghai's unique location, it is sensitive to sea level rise and land subsidence, which are directly related to urban flood protection. Flooding of Shanghai can be caused due to the following three factors: storm surge by typhoon, high tidal levels and sustained higher water levels in the Yangtze River and the Huangpu River as a result of the higher tide in the mouth of the river during the rainy season. Typhoons moving towards Shanghai not only bring high storm surges but also heavy rainfalls and strong winds to the area.

In response to the potential floods caused by typhoons to Shanghai city, China, several proposals emerged to protect the region in the first design level. By using Multi Criteria Analysis, one of them is selected as the protection system. A challenge for this 15 km long barrier system is to cross Yangtze Estuary, including two islands and three channels at the Yangtze River mouth. A storm surge barrier is required to close the coastal spine and prevent storm surges in the Yangtze Estuary. This thesis, based on a system engineering approach, presents the design process and a feasible design for the storm surge barrier at this specific site.

The design level 2 starts with a framework consisting of a program of requirements and boundary conditions. According to the requirements the barrier system is divided in two sections: a navigational section spanning the deeper section of the waterway in the South Channel allowing the passage vessels in accordance of '*Deepwater Channel Regulation Project*'; and an environmental section which aims to preserve the ecosystem. The objective of this thesis is to make a conceptual design of the navigational section to satisfy two requirements. Firstly to enable free navigation under normal conditions, the minimum opening width is calculated as 172 meters. Secondly is to sufficiently reduce the effect of storm surges with a probability of occurrence of once in one thousand years, the maximum water level rise inside the estuary is determined as 3.5 meters.

The first step is to consider the barrier as a whole system. A modified 'storage-basin method' considering the river discharge from the Yangtze River is applied. It shows the large retention capacity of the Yangtze Estuary allows the barrier not to be fully retaining. A closed barrier, with a continuous gate top level of WD +5.5m, reduces the surge level sufficiently (3.6 m higher than normal water level). With this retaining height along both sections, the barrier is overflown with an overtopping rate of 9.49 l/s/m, but the basin's retention capacity is ensures the flood behind the barrier is acceptable.

Considering the environmental requirements, it is decided to keep at least 60% of the original flow area open, resulting in minimum 8.5 km long environmental section. The vertically lifting

gates are selected as environmental gates, because they are feasible for large spans and suited to reverse differential head and reverse flow during operation. The main global structural dimensions are determined, for both full-retaining and limited-retaining lifting gates.

Then the design goes into the navigational section in the South Channel. It can be either open-closable or permanently open. The main functional requirements for the navigational section are as follows:

- For navigation:
  Minimum opening width of 172 meters, minimum channel depth of 16.25 meters;
- For safety inside the Yangtze Estuary:
  Maximum opening width of 375 meters.

From then on, two solutions are studied for part of the barrier system in the South Channel. One consists of a 180 m long barge gate for the navigational section and 44 lifting gates for the environmental section. The barge gate is suitable for the wide openings, provide unlimited air draft and in the floating situation doesn't transfer too much loads to the foundations. The barge gate is designed in more detail, including the materials, supporting structures, operation system, foundation and bed protection etc. The other option consists of a 375 m long permanent opening for the navigational section and 42 lifting gates for the environmental section.

Table. Summary two options of barrier design in the South Channel

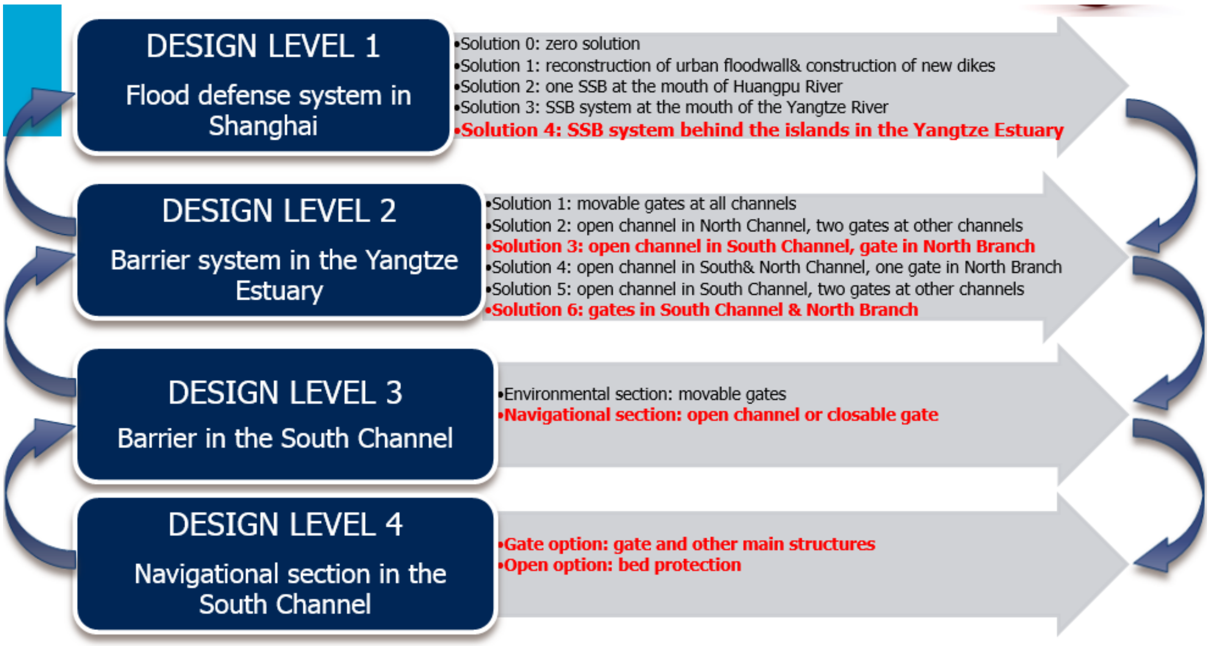
	Navigational gates		Environmental gates	
	Parameter	Value	Parameter	Value
GATE OPTION	Gate type	Barge gate	Gate type	Vertically lifting gate
	Gate material	Prestressed concrete	Gate material	Steel
	Opening width	180m	Opening width	80m
	Number of openings	1	Number of openings	44
	Top level closed gate	WD+5.1m	Top level closed gate	WD+5.1m
	Sill level	WD-16.5m	Sill level	WD-12.5m
	Gate height	21.6m	Gate height	17.6m
	Navigational gates		Environmental gates	
	Parameter	Value	Parameter	Value
OPEN OPTION	No gate		Gate type	Vertically lifting gate
	Permanently opened during all conditions		Gate material	Steel
	Opening width	375m	Opening width	80m
	Number of openings	1	Number of openings	42
	Sill level	WD-16.5m	Top level closed gate	WD+7.0 m
			Sill level	WD-12.5m
			Gate height	19.5 m

The main difference between the gate option and open option lies in the bed protection behind the navigational section. With a closed barge gate, the current field is much softer than open option during storm conditions. Heavy bed protection with large rocks and several filter layers is required for the open option, to sustain the high current velocities up to 7.8 m/s. Applying Pilarczyk equation, the downstream protection length behind the opening should be minimum 400 meters.

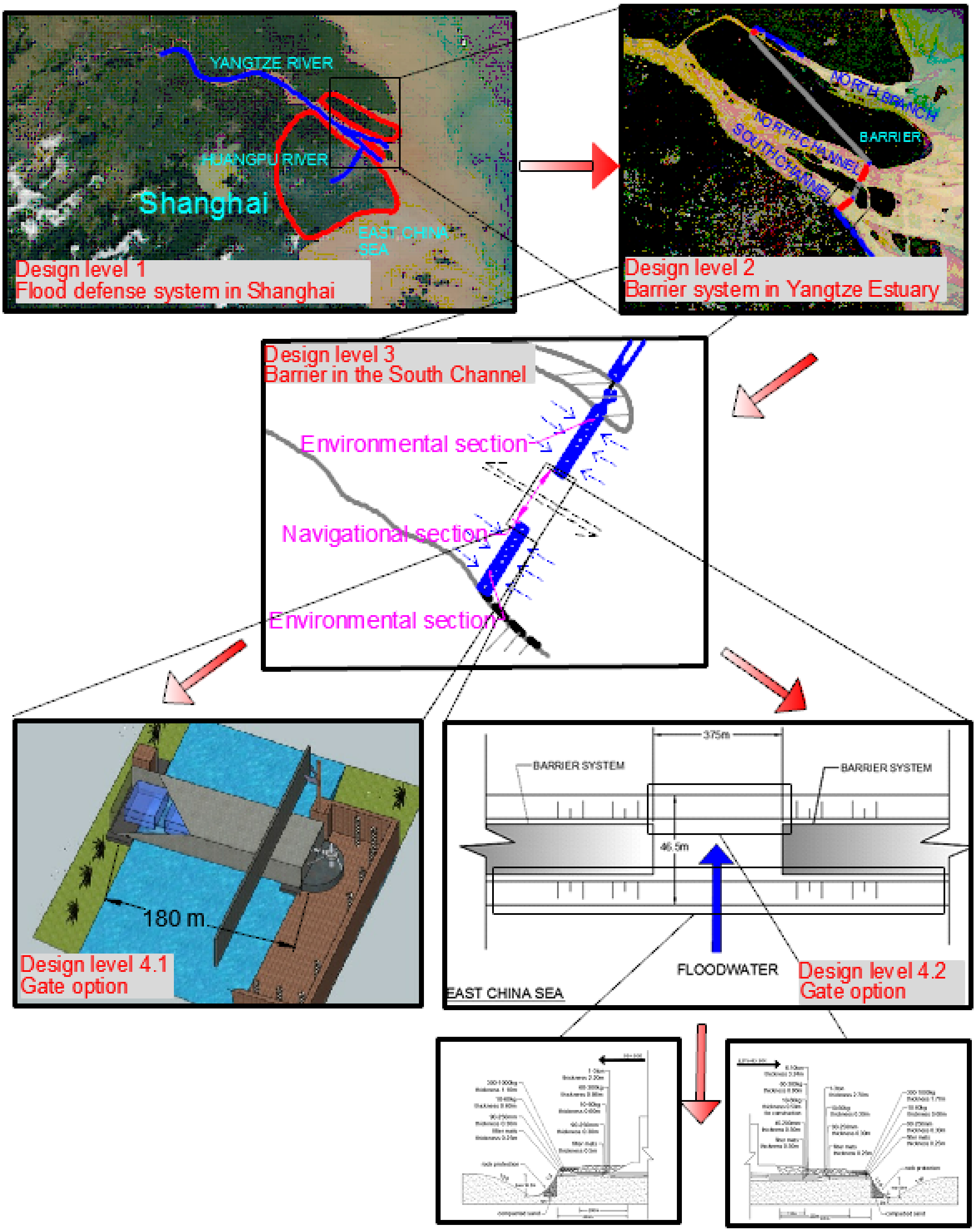
The report proceeds with design level 4, studying these two options in more detail with costs estimation.

The estimation is performed focusing on the main parts of the barrier systems. The results reveal the total costs of open option are around 1% higher. That is because much heavier bed protection work is required to sustain large current velocities behind the opening. This caused the total costs of the open option to increase a lot, which even encounters the reduced costs due to less required gates.

SYSTEM ENGINEERING APPROACH



PROJECT MAP



YANGTZE RIVER
HUANGPU RIVER
Shanghai
EAST CHINA SEA
Design level 1
Flood defense system in Shanghai
NORTH BRANCH
BARRIER
NORTH CHANNEL
SOUTH CHANNEL
Design level 2
Barrier system in Yangtze Estuary
Design level 3
Barrier in the South Channel
Environmental section
Navigational section
Environmental section
180 m
Design level 4.1
Gate option
375m
BARRIER SYSTEM
BARRIER SYSTEM
46.5m
FLOODWATER
EAST CHINA SEA
Design level 4.2
Gate option

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1. INTRODUCTION

This chapter includes a short background about the Shanghai city. Also general information about the current report is listed. After review of the previous studies, the main problem is pointed out and the research questions are kept down. The desired design approach for the project is also depicted in the last part of this chapter. Finally, the structure of the report will be descried in order to offer a clear view.

1.1 Background

The background information includes a shot description of Shanghai and its surroundings.

1.1.1 Description of Shanghai

Shanghai is a coastal city in China situated on the central China's eastern shores at 30.42' - 31.48' N. It lies on the southeastern frontier if the Yangtze River Delta and therefore contains lots of rivers, canals, drains and lakes, and is bounded to the east by the East China Sea. The city is one of the four direct-controlled municipalities of China, with a total population of 14 million permanent and 3 to 4 million transient inhabitants. The historic center of the city, the Puxi area, is located on the western side of the Huangpu, while the newly developed Pudong, and was developed on the eastern bank. At present, Shanghai is the Asian financial center, and a transport hub with the world's busiest container port.

Figure 1- 1. Location Map of Shanghai Region (Source: (EzilonMap))

Due to its location on the deltaic deposit of the Yangtze River, the thickness of Quaternary deposit is about 300 m. Most of Shanghai is constructed on early strand plains, which are primarily constructed by waves and tidal currents from run-off washed down the Yangtze River. However, due to flood control structures built upstream on the Yangtze River, fresh layers of sediment are not replenishing surface areas lost by subsidence.

In shanghai urban area, the average ground level is 4.5 m (Wusong datum (WD)[1]). Due to global climate change, the absolute sea level change in Wusong is forecasted to rise at a rate of 2.5 mm/a in the period 1999-2030, and continue increasing to 5.0 mm/a in the following two decades. The absolute rising height is 5 cm, 10 cm and 20 cm in 2010, 2030 and 2050(Yin & Xu, 2013).

Land subsidence in Shanghai is also quite evident like many other big cities. Increasing investment of infrastructure utilities caused dramatically development of construction of high-rise building in the last 20 years, resulting in land subsidence. Along with that, excessive pumping of ground water exacerbated the subsiding of land, which would be a serious problem to protect the city from flooding. According to estimate, the mean land subsidence will be 10cm, 15cm and 17.3cm in 2010, 2030 and 2050 respectively(Shen, 1997) .

1.1.2 Shanghai Water System

Like many other coastal cities along China's coastline, Shanghai is vulnerable for coastal floods due to a combination of typhoon-induced storm surge from the East China Sea, high river flow on the Huangpu River and heavy rainfalls. The city's main river, the Huangpu River, is the main waterway as well as the main flood diversion route to the westward-located Tai Lake (see Figure 1- 2). The river menders through the urban Shanghai, and links Tai Lake and the mouth of the Yangtze River. It also drains 70% to 80% of the Tai Lake Basin(Zhao & Deng, 2005). The Tai Lake Basin covers an area of 36,900 km^2, about 0.4 % of China's land but contributes for 21% of China's GDP. The urbanization level of the lake basin ranks the first in the entire country. The water surface of the Tai Lake totals 2,338 km^2, is the third largest freshwater lake in China and the key source of water supply to the region(Ren et al., 2003).

[1] Which is 1.924 m lower mean sea level of China Yellow Sea; Hereafter, geographic elevation and water levels in the river and Sea are adopted at Wusong Datum.

Figure 1- 2. Surrounding Water Bodies of Shanghai

The water system of Shanghai area is shown in Figure 1- 2 and composed by:

- Yangtze River Estuary
- Huangpu River
- Tai Lake Basin

Yangtze River Estuary

The Yangtze Estuary (90 km wide, seperated by the Congming Island) is the lowest part of the Yangtze River, the third longest river in the world (6,300 km)(OCEANA). The estuary carries an average of 30 million liters of water per second into the East China Sea; its average depth is 7 m, and the average tidal range at its mouth is 2.7 m. The annual discharge of the Yangtze River is around 29,000 m^3/s. During the rain season from May to October, the average discharge the is 41,100 m^3/s. It supports large numbers of fish and birds, although fish stocks have declined over the past 20 years due to overfishing and pollution. The estuary's waters may be fresh, brackish, or salty, depending on the season.

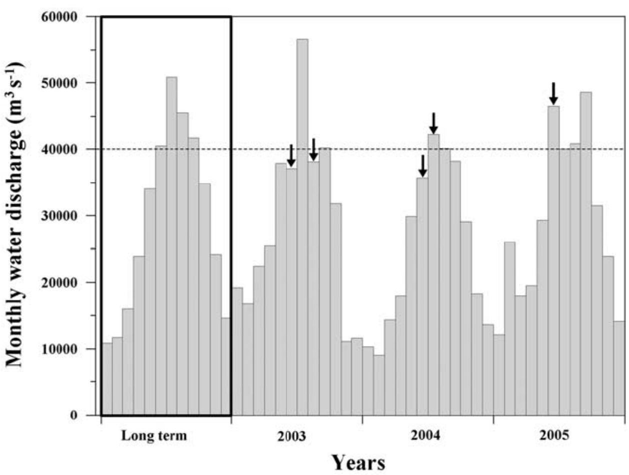


Figure 1- 3. Monthly water dishcharge rate (m^3s^{-1}) for long-term averaged values(in black frame) and for 2003, 2004 and 2005 at Datong Station, the seaward-most station of the Yangtze River. (Source: (Yang et al., 2006))

Figure 1- 4. Yangtze River estuary

Huangpu River

The Huangpu River originates links the Tai Lake via the Taipu River and menders through the downtown area of Shanghai. The river is the main waterway as well as the main flood diversion with the Yangtze River. It is the last significant tributary of the Yangtze before it empties into the East China Sea, which flows through the urban area of Shanghai (see Figure 1- 5). The Shanghaiese affectionately call it the "mother river" as it serves the multiple purposes of water supply, water disposal and transportation.

Figure 1- 5. Satellite Map of Huangpu River

The Huangpu River is 113.4 km long and around 750 m wide at the mouth at Wusongkou and average 360 m elsewhere. The river is also the main water outlet to the Tai Lake Basin, with depth ranging from -15 m to -8 m. It is born from the convergence of Xietang and Yuanxiejiang creeks from Tai Lake, and the Damaogang Creek from Zhejiang Province at Mishidu in Songjiang Country. The Huangpu River then winds sic counties (Qingpu, Songjiang, Fengxian, Shanghai, Chuansha, Baoshan) and it joined by over 200 branches, the largest of which are the Dianpu River and Suzhou River. The Huangpu River then proceeds through downtown Shanghai and finally drains into the Yangtze at the estuary of Wusong mouth. The 30 kilometer section of the Huangpu River that runs through Shanghai's downtown serves to geographically divide the city map.

Figure 1- 6. Hydrological Observation Station along the Huangpu River in Shanghai

The Huangpu River is governed by semi diurnal tide in the East China Sea. At Wusongkou Station, in the river mouth the mean low and high water levels are 1.03 m and 3.24 m respectively. The flood period is 4 hours and 33 minutes. The average annual tidal rang is 2.27 m(Nai, 2003).

The long term annual average discharge is 304 m^3s^{-1} measured at Mishidu station in the rain season from May to September and 328 m^3s^{-1} for the remaining months. The upstream water inflow into the Huangpu River varies with the hydrological year(Yuan, 1999).

Tai Lake Basin

The Tai Lake Basin is the third largest fresh water lake in China and the main water supplies to Shanghai. The lake is mainly supplied by rivers and streams draining the higher grounds to the west. The basin has a yearly averaged water deficit of 2 billion m^3 for normal years and 12 billion m^3 for dry years. The volume of water in Tai Lake varies greater from year to year and thus its significance in providing water to Shanghai also varies. On average, however, Tai Lake provides 16.9% of surface water resources in Shanghai.

The Taihu Basin covers an area greater than 36, 000 km^2 within the Yangtze delta region in east China, spanning the administrative districts of Shanghai, Jiangsu, Anhui and Zhejiang (Figure 1- 7). The Basin is faced with flood risks arising from a large number of physical drivers. Pluvial flooding (monsoonal rainfall), fluvial (river) flooding, interurban flooding (associated with urban drainage systems) and coastal flooding (storm surges) are exacerbated by the low-lying topography (land surface elevations).

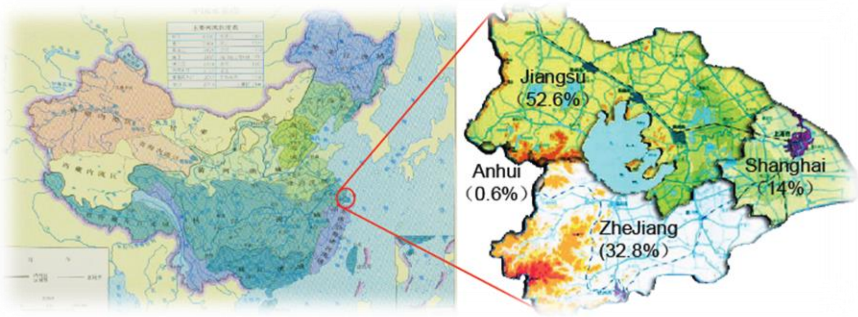


Figure 1- 7. Location Map of Tai Lake Basin (Source:(Harvey, Evans, Thorne, & Cheng, 2009))

1.2 Problem Definition

Due to Shanghai's unique location, it is sensitive to sea level rise and land subsidence, which are directly related to urban flood protection. Flooding of Shanghai can be caused due to the following three factors: storm surge by typhoon, high tidal level and sustained higher water levels in the river as a result of the higher tide in the mouth of the river during the rainy season. Typhoons moving towards Shanghai not only bring high storm surges but also heavy rainfalls and strong winds to the area. These intense rains temporarily increase the runoff into Huangpu River substantially and may cause flooding of the Huangpu River when this occurs during barrier closure.

Hence, the main problem of this thesis is defined as:

"*What is the most suitable, reliable and economical option to protect Shanghai from flooding, and the conceptual design for the navigational section?*"

1.3 Objective

To solve this problem, the main objective of this study is to provide a preliminary design of the main structural parts of a storm surge barrier in either the Huangpu River or the Yangtze River.

Also, the following sub questions are to be answered:

- What is the current situation in Shanghai and what is needed for the future?
- What is the best barrier location in this complicated water system?
- What are the main requirements and dominant boundary conditions?
- What kind of barrier suits the specific site conditions best?
- How large should the gate openings be?
- If gate(s) chosen for the navigational section, what kind of gate is the best option? Or a combination of several advantageous gate types?
- What are the main load(s) on the gate(s) when the barrier is in operation?
- Is it possible to keep it permanently open in certain section of the system?

1.4 Work Approach

This master thesis is based on a preliminary design of a storm surge barrier to protect Shanghai from flooding, part of the flood protection system in the Yangtze Estuary. This study focuses on flooding induced by severe typhoons from the East China Sea, combined by the influence of high river discharge from the Yangtze River.

Shanghai flood protection system is a huge and complex system to design. A design methodology applied within this research is called “System Engineering”, which gives an insight in the complexity of the object, which has to be designed. The whole design process is divided into several levels, within which a decision is made per design level. A cyclic, iterative process form large to small-scale is used in this methodology.

The main thesis will have its focus on the following design levels:

<u>Level 1- Flood defense system to protect Shanghai from flooding</u>

Determine the requirements for the current situation and what is needed for the future. Determine the master plan for the whole system by evaluating several alternatives.

<u>Level 2- Barrier system in the Yangtze Estuary</u>

Further investigation into the selected master plan with floodgate(s) is performed, by providing different functional sections in the Yangtze Estuary. No flooding is allowed in the estuary or to upstream cities in the design storm.

<u>Level 3- Barrier in the South Channel</u>

Determine the distribution of different sections according the local boundary conditions. Determine the most cost-efficient design by evaluating several options.

<u>Level 4- Open or closable navigational section in the South Channel</u>

Whether open or closable navigational section is determined. The comparison is mainly based on a rough cost estimation on the pre-feasibility design level.

2. SHANGHAI AND FLOODS

Shanghai experienced hundreds of flood disasters in 2,000 years of recorded history. The storm surge accompanying the typhoons can easily swell the tide in the Huangpu River to breaching the flood defenses along the river with inundation or large parts of the urban Shanghai area.

2.1 Historical View of Flood Events

According to the historical record, flooding is mainly caused by the high storm surge from the East China Sea or high discharge from Yangtze River. In addition, the torrential rainfall can also lead to water logging in this low-lying area in Shanghai. The main historical flood events since 1900 are show in the table below (see Table 2- 1). Wang & Plate (2002) have also studied recent destructive typhoon disasters formed in Shanghai (see Table 2- 2). The data listed are all collected at the Huangpu Park Gauge Station.

Table 2- 1. Historical Record of Flood Events Since 1900

Year	Date	Highest Water Level (m)
1905	01/09	5.24
1914	24/28	4.73
1921	20/08	4.88
1931	25/08	4.94
1933	18/09	4.86
1939	30/08	4.70
1949	25/07	4.77
1956	02/08	3.99
1962	02/08	4.76
1974	20/08	4.98
1981	01/09	5.20
1989	04/08	5.02
1997	16/08	5.72
2000	31/08	5.70
2000	14/09	5.22
2005	06/08	4.94

Table 2- 2. Recent Flood Caused by Typhoon in Shanghai

Year	Date	Typhoon
1997	16/08	Typhoon Winnie
2000	31/08	Typhoon Papain
2000	14/09	Typhoon Sangmei
2005	06/08	Typhoon Masha
2012	08/08	Typhoon Haiku

According to the report "*Analysis of Historical Flood Events in Shanghai*" by the Shanghai Water Authority, huge storm surges formed 17 times since year 1900. The Yangtze River flood killed 145,000 people and around 28.5 million were affected in the year of 1931.

One destructive typhoon occurred on Augest 18th, 1997, named Typhoon Winnie. It was designated as a Category V "Super typhoon" when it brought the highest recorded water level, 5.72 meters higher than the normal water level, and storm surge 1.45 meters when it made landfall just south of Shanghai. Flooding during that typhoon killed 3,500, with monetary losses estimated at € 28 billion. More recently, in the year 2012, when typhoon Haikui came, more than 3.2 million residents were affected and 2,900 houses were flattened, and 30,000 businesses suspended production. About 220 roads and 770 electric lines were desyroyed. Typhoon Haikui forced Shanghai to raise its highest-level alert as a wake.

2.2 Typhoons in Shanghai

Typhoons coming from the western North Pacific (WNP) mostly hit Shanghai during the boreal summer (June to September) every year. This name of Typhoon comes from a Chinese word Tai-fung meaning a great wind from the sea. They always come accompanied by heavy rains. The Japan's National Institute of Informatics (NII) gives the definition of Typhoon as a tropical cyclone (hereafter referred as TC) with the maximum wind of 34 knots[2] or higher. Therefore, it is quite necessary to understand the characteristics of tropical cyclones influencing Shanghai and associated storm surges will be analyzed in this section.

Refer to Appendix D for more information on typhoon characteristics.

The only good thing typhoons can bring is abundant rain. On the other hand, the damage caused by typhoon, is countless. As is illustrated in section 2.1, several typhoons hit eastern China's coast every year, bring heavy rains and storm surges, causing loss of human life and destructing properties, causing great damage to the community. For example, Typhoon Haikui in 2012, which was developed in the northwest Pacific and attacked Shanghai, killed 23 people. Typhoon Haikui made landfall in Zhejiang Province south to Shanghai, after Shanghai officials have moved 374,000 people to emergency shelters. The storm had cut off electricity to nearly 400,000 households in Shanghai. The economic loss amounted to 3 billion euros.

Frequency

On average, 38 tropical cyclones have developed in the Northwest Pacific since 1949 every year. Annually 3.2 tropical cyclones have effects on Shanghai. The frequency varies between years, depending on the global atmosphere circulation (see Figure 2- 1).

[2] Knot is a unit for speed. One knot means a speed of moving one nautical mile (nm) in one hour.

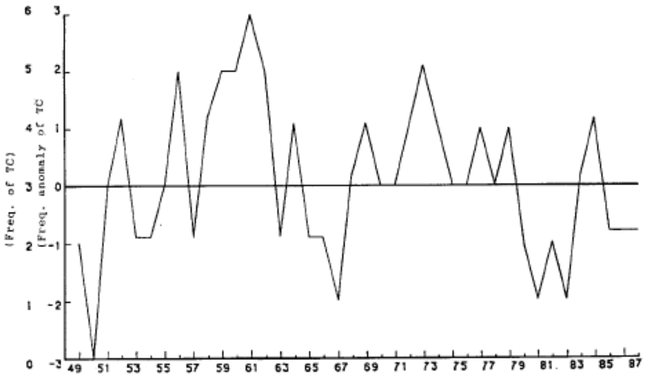


Figure 2- 1. The Frequencies of Tropical Cyclones Affecting Shanghai During the Period 1949-1988 (Source: (Qin & Duan, 1992))

In general, the tropical cyclones can happen all the year around. However, the tropical cyclone season ranges from May to November in Shanghai, and nearly 90% of the affecting cyclones occur from July to September. Relatively high frequency appears in August.

Spacial Distribution

Unlike other natural disasters, a tropical cyclone is a moving system, the rainfalls, storm surges and strong winds move with them. The damage depends mostly on the landfall of a tropical cyclone. The landfall location can be forecast accurately with satellites, by the means of monitoring the movement of the tropical cyclones.

Chen (2000) has analyzed the tropical cyclones data for 30 years since year 1949, there is an average of 9 landfalls annually. It is rated the highest tropical landfall for a country around the world. However, the Shanghai area has the lowest frequencies along the coast, only 7 landfalls occurred during the last half century. Some tropical cyclones do not make landfalls, they just pass through the area. The majority of floods in Shanghai were caused by this kind of tropical cyclones.

Speaking of the moving direction, tropical cyclones coming from the Northwest Pacific normally turn westward, driven by easterly trade winds towards China’s coast, and then migrate into higher latitudes. From Figure 2- 2, the tropical cyclones can be further divided into four types according to their tracks:

- Type 1. recurving in the open sea;
- Type 2. recurving offshore;
- Type 3. landing on the coastline;
- Type 4.turing towards the west. During those types, the last one has the highest frequency of nearly 50%.

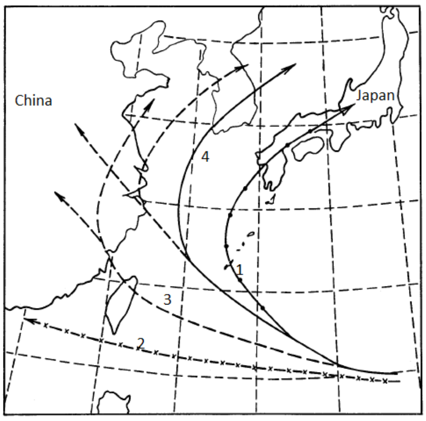


Figure 2- 2. Four Types of Tropical Cyclones' Tracks (Source: (Tang & Wei, 2013))

Associated Hazards

There are three major hazards that typhoons bring to Shanghai area, including storm surges, torrential rainfall and strong winds. As typhoon is a moving system, these hazards move along with typhoon and affect the passing area.

Typhoons bring along storm surges, which is created by the low atmospheric pressure in center of typhoon combined with the surface wind stress. The most dangerous storm surges are caused by the second type of tropical cyclones recurving offshore (as stated in section Spatial Distribution). Qin (1992) has analyzed the data over the period between year 1949 to 1879, during which there were 67 storm surges with surge heights larger than 25 cm. Among those storm surges, almost 90% of them occurred within July, August and September. Around 50% of the storm surges were induced but tropical cyclones with offshore recurving tracks, while only one quarter were induced by tropical cyclones making landfall at around 30 ^{o} N.

Secondly, torrential rains induced by typhoons can cause waterlog, particularly in the most dense area of Shanghai. Due to continuous land subsidence, this kind of hazard has been deteriorated. Regarding to the strong wind brought by typhoons, the most severe damage occurs only when the typhoons hit Shanghai directly.

3. FLOOD DEFENSE SYSTEM IN COASTAL CITIES

The sea not only provides benefits, but also poses hazards to human society. In this chapter, three coastal flood defense systems are illustrated: the south-western delta area of the Netherlands, New Orleans in the United States, and Shanghai in China. For more details on these reference flood defense systems, Appendix A is attached.

3.1 The Delta Plan in the Netherlands

The Delta Plan is a series of construction projects to protect the delta area from flooding in the Netherlands. More than half of this country is below the storm surge level; the south-western part, the most densely populated area, is protected by dunes, dikes and other hydraulic structures.

The Dutch have been fighting against the "Water Wolf" for centuries, as they call it the ancient enemy. In the past, the delta area was flooded several times. Among those floods, the most recent and the most destructive is the flood in 1953 (see Figure 3- 1), during which more than 1,800 people died. As a wake, the Dutch people realized such a disaster must be prevented in the future and forever. Immediately after the flood, the Delta Plan was drawn up, a plan in which the south-western part of the Netherlands is cut off from the sea by various structures. Only the Western Scheldt remains open to get access to Antwerp Harbor and the Rotterdam Waterway.

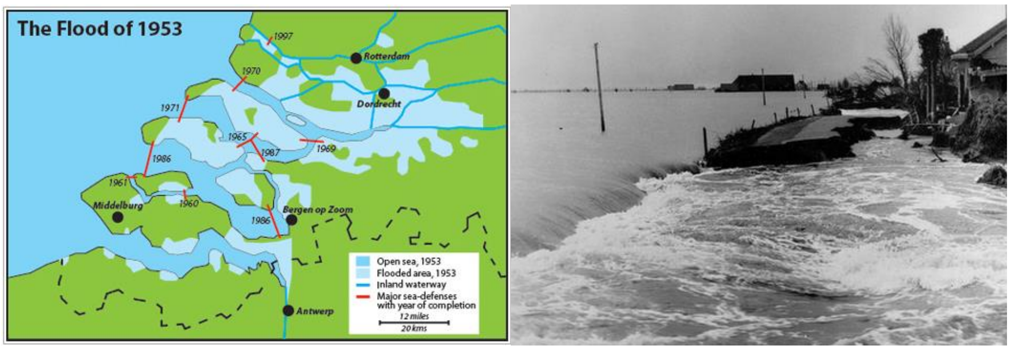


Figure 3- 1. Situation Flood 1953, the Netherlands

Storm Surge Barriers

At present, there are 13 closure dams and storm surge barriers constructed (see Figure 3- 2). The Hollandse Ijssle storm surge barrier was the first project of the Delta Works. Three years later, two more dams named the Zandcreek and the Veerse Gat separately, were finished. The two dams cut off the flow from the North Sea and formed the Veerse Meer. The Haringvliet Dam was closed in 1971, as a big discharging sluice complex, regulating the water from the Rhine and Maas. The Brouwers Dam and Grevelingen Dam were constructed in the subsequent years. The Maeslant barrier, the most popular one, was built with two steel sector gates, creating a new waterway in Rotterdam.

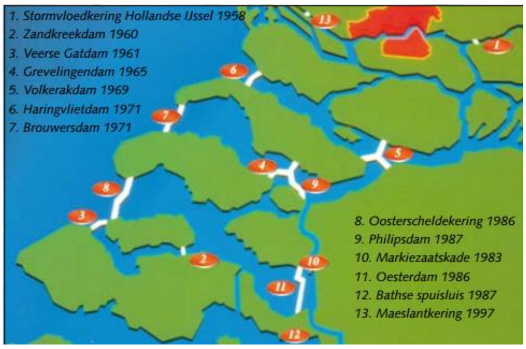


Figure 3- 2. Present Storm Surge Barriers in the Delta Area (Source: (Rijkswaterstaat))

Dike Rings

Major area should be protected by a ring of primary sea defenses. This kind of sea defense is called dike ring. The Delta Plan subdivided the whole country into “dike rings”, each of which has a different safety level, according to flood risk assessment. Two important dike rings 14 (Central Holland) and dike ring 15 (Krimpenerwaard) are shown in Figure 3- 3 (Welsink, 2013).

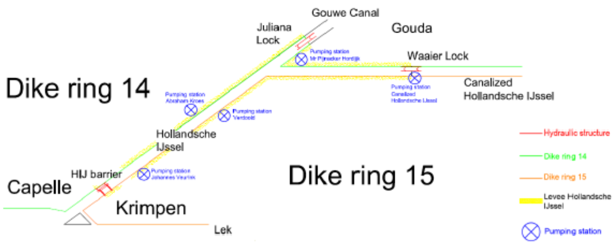


Figure 3- 3. Schematic System of Dike 14 and Dike 15

3.2 New Orleans Hurricane Protection System

New Orleans is an American city situated in the southeast Louisiana, at a point where the Mississippi River flows into the Gulf of Mexico. It is located in the low-lying tidal area, surrounded by water. Louisiana’s coastal zone includes almost half of U.S. coastal wetlands and ¼ of all U.S. wetlands. With a dense population and well developed industries, the New Orleans also has the problem of ground settlement and coastal erosion as other big coastal cities. The city of New Orleans has been against flooding of Mississippi River and associated storm surges brought by hurricanes every year.

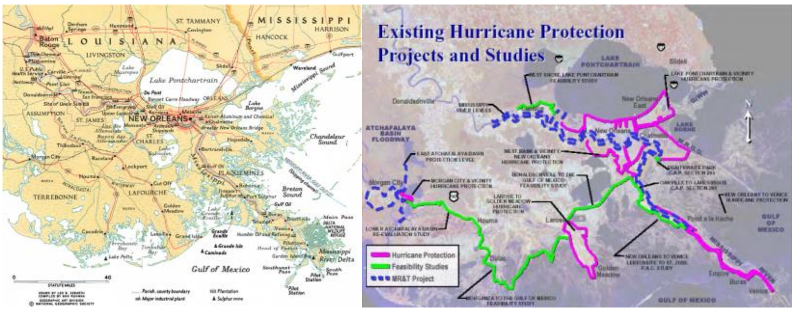


Figure 3- 4. Modern Mississippi River Delta in the Vicinity of New Orleans

To ensure safety of New Orleans, the levees came into the hydraulic engineers' mind. Levee construction started in the 18th century, and were heightened and strengthened in the following years.

In a sever flood in 1927, New Orleans avoided inundation thanks to the levees(Huang, 2007). The complete flood protection system was set up by the US Army Corps of Engineers in the 1950s, however, the high water still threatens the city now. After the flood in 1927, as a wakeup call, the federal government, the flood control system formed, carried out the Flood Control System.

3.3 Shanghai Flood Defense System

Shanghai region has a huge complex water system. There are about ten rivers besides Yangtze River and Huangpu River, contributing to the local drainage system. However, the Huangpu River, as the main river in Shanghai, drains 40% of water in the Tai Lake. If typhoon, storms and astronomically come at the same time, the water from upstream will go down. The water level will then rise rapidly, causing severe flooding.

According to the National Standard *"Standard for Flood Control, GB50201-94. (1995)",* the primary flood retaining structures around Shanghai have to provide full protection against floods with a return period of 1,000 years. One important point of difference should be made, as the hydrological conditions for each location is not similar. This difference has consequences for the design of a protecting system. As in the Netherlands, a safety level of 1/10,000 [1/year] is used in the central coastal area, with a lower design level in the rural area. Though the population is much denser of Shanghai than that of Rotterdam, land in Shanghai is still 2 to 3 meters higher than the mean sea level. Although not further investigated, applying this criterion to Shanghai is not expected to be cost efficient.

To protect Shanghai against flood disasters, the local government has reinforced and extended its four defense lines (see Figure 3- 5):

- The sea dike. It is the first line, which can withstand several tropical cyclones;
- Urban flood wall. It extends to 315 km through the urban area, with its sluices and gates along the Huangpu River;

- Smaller flood walls along other rivers in Shanghai city;
- The network of underground drainage system to prevent water logging of urban area.

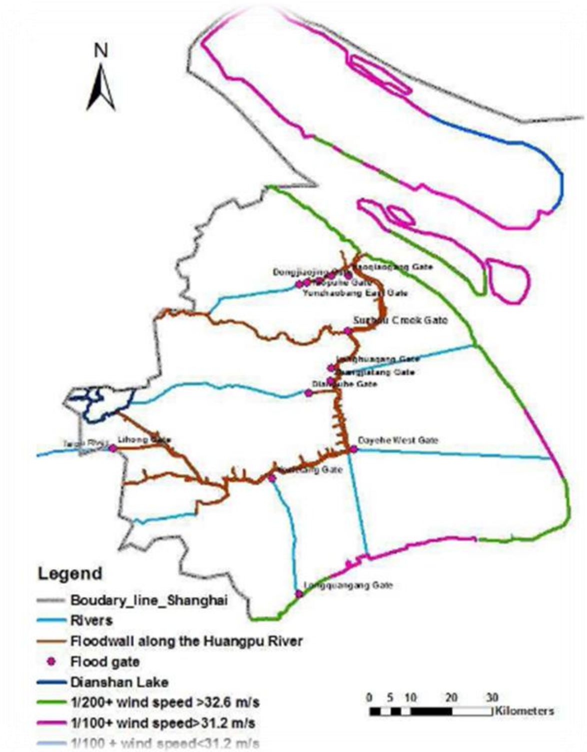


Figure 3- 5. Flood Defense System in Shanghai (Source: (Ke, Jonkman, Dupuits, & Kanning, 2013))

3.4 Conclusion

As explained in the above sections, big coastal cities often have quite similar flooding problems: sea level rise, land subsidence and dense population. This has led to global studies in how to protect these cities in such complex water system. The Dutch and US engineers have successfully set up their own flood control systems, which always consist of multiple defense lines, including barriers, dikes, levees and various non-engineering solutions.

However, Shanghai flood defense system only has sea dikes, flood walls and urban underground drainage system, which makes Shanghai a weak point to fight against the potential floods due to changing boundary conditions. Four methods are proposed to increase the whole system capacity.

Heightening or reconstruction of existing walls along the rivers, or closing down the tidal inlets by constructing a storm surge barrier can either be a solution. However, rebuilding the floodwalls could be quite expensive, difficult and less efficient due to its irregular distribution and poor management. Although construction of a storm surge barrier could be a better choice, the final choice still depends on various factors, like local conditions and requirements etc.

4. STORM SURGE BARRIERS

A storm surge barrier is a partly movable barrier constructed in an estuary or river branch, which can be closed temporarily when a storm surge is expected. It is designed to protect the area behind the barrier, from flooding induced by storm surges. Under normal circumstances, the storm surge barrier allows water to pass, when a storm surge is expected, the barrier will be closed.

As such measures are always of great cost, storm surge barriers are often constructed after a sever flood. For example, the Delta Works in the Netherlands after the flood in 1953. Barriers are currently being designed in New York after the Hurricane Sandy as a wake-up call for the US society. To design a barrier, one often begins with an analysis of the functions and requirements of a barrier. Besides its basic function of retaining a storm surge, aspects such as reliability, maintainability, durability, constructability, environmental impact and economical aspect can play an important role in decision-making process. This chapter gives general information about three barriers in the Netherlands and South Korea; more details are included in Appendix B. A proposed idea on a barrier in the Huangpu River is also explained in this chapter.

4.1 Projects Review

In 2006, the PIANC Working Group 26 of the Inland Navigation Commission, published a report named "*Design of Movable Weirs and Storm Surge Barriers*", which presents an inventory of barriers located around the world. However, in coastal cities like Rotterdam and Shanghai, the barriers are required not to obstruct shipping under normal conditions, except their basic storm surge retaining function. In this section, three special existing storm surge barriers are reviewed, including their barrier locations, dimension, functions and construction methods. At the end, details about several proposed barriers in Shanghai by the local government are also descripted.

4.1 Haringvlietdam

Delta plan is formed by the big discharging sluices complex in the Haringvliet, which was finished in the mid-sixties. The Haringvlietdam is located between Goeree- Overflakkee and Voorne Putten in the western-south of the Netherlands, see Figure 4- 1. Except the basic flood protection function, it also works as the drainage between the Rhine and Meuse River with the North Sea. The execution of this dam started with the pile foundation, then the sluices were constructed. The Haringvlietdam is made of concrete blocks, which were dumped on the sea bottom from the cable cars. The properties of the Haringvlietdam are listed in

Table 4- 1.

Figure 4- 1. Overview of Haringvlietdam and its Location (Rijkswaterstaat)

Table 4- 1. Properties of the Haringvlietdam

Year of construction	1957-1971
Length dam	1000 m
Dimension	width 65 m (crest) to 330 m(at bed level)
Soil condition	Clay and soft sandy layers
Max. wave height	4 m
Discharge	25,000 m3/s

Regarding to the construction process, the sluices were first constructed in the middle of the Haringvliet. A 3 m thick concrete floor were constructed on the pile foundation, see Figure 4-2. The opening is 56 m wide. The opening was designed to allow the discharge of ice during wintertime. Each opening consists of two gates. The inner gate can take the excess volume discharge during storm surges; the seaward one can break the waves from the sea. The bottom of the sluice is protected with a concrete slab.

The large concrete blocks were dropped into the water by the cable car. To close the southern gap, sand was raised on the section of the dam, until a dike was formed. The remaining northern gap was much harder to close. More than 100,000 large concrete blocks (each weighs 2,500 kilograms) were plunged by cableways. The holes between the concrete blocks were filled with sand. During the construction of dam, the sluices were open to allow tidal movements. After the northern part was completely closed, the sluices were closed. Then the Haringvliet became a lake.

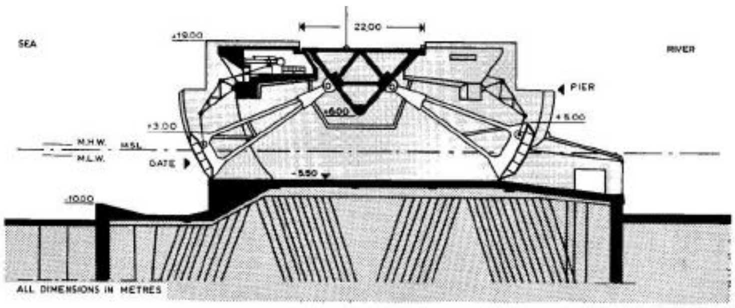
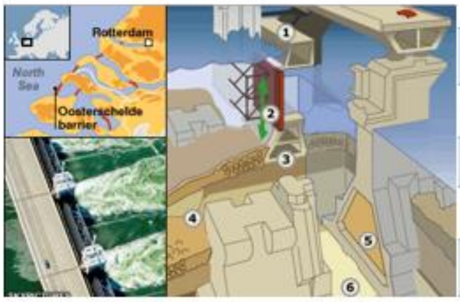


Figure 4- 2. Cross Section of Haringvliet Sluice (Ferguson, Blokland, & Kuiper, 1970)

4.2 Eastern Scheldt Storm Surge Barrier (Oosterschelde)

The Eastern Scheldt (Dutch: Oosterschelde) Storm Surge Barrier is the major flood protection barrier in the Netherlands. The Eastern Scheldt Storm Surge Barrier is the biggest structure of the Delta Works, which separates the Eastern Scheldt and the North Sea. It closes off the Eastern Scheldt Estuary in case of a high tide, but remains open under normal circumstances to maintain salinity.

When the Haringvliet Dam (see Section 4.1) was nearly finishing, the largest, last and the most complicated part of the Delta Plan were being prepared, to build a dam across te mouth of the Eastern Scheldt. The storm surge barrier in the Eastern Scheldt was commissioned by the Rijkswaterstaat[3] and built by a joint venture by many contractors. At first, the plan was to build a complete closure dam. Three islands were constructed: Roggeplaat, Neeltje Jans and Noordland. However, due the political and social pressure, the plan was changed to preserve the Eastern Scheldt ecosystem.



Properties	
Execution	1978-1986
Length barrier	2800 m
Height piers	30-40 m
Length gates	45 m
Cost indication	€ 1.3 billion

Figure 4- 3. Eastern Scheldt Storm Surge Barrier and the Properties (Source: (de Gijt & van der Toorn, 2013))

For the construction of the barrier, it was decided to not build it in situ, in order not to obstruct the local natural system. In case the barrier would be executed in situ, equipment would have experienced more troubles from the tidal currents and other unfavorable weather conditions. Thus, the barrier was built with pre-constructed elements, including concrete piers, concrete sill beams, concrete upper beams, and steel lifting gates. The sill and bed protection have been built with riprap combined with asphalt. The gates are lifted by hydraulic cylinders. 62 lifting gates were constructed. The heights of gated are between 5.9 m and 11.9 m, weights between 300 and 535 tons, at different locations. The horizontal force is transferred to the supports in the piers. There are 65 piers in total, the maximum height of pier after completion in tidal channel is53 m. Due to the combination of gates and fixed concrete sill beams, shipping is not allowed. To overcome this advantage, a separate lock is provided. The barrier can be closed within one hour. The barrier is subjected to heavy wave loads (design wave condition: wave height 5.8 m and wave period 10 seconds). With the

[3] Rijkswaterstaat is part of the Dutch Ministry of Infrastructure and the Environment, responsible for public works and water management.

Eastern Storm Surge Barrier, the chance of a flood in this area has been reduce to one in every 4,000 years.

4.3 Saemangeum Dike

The Saemangeum project is the largest land reclamation in the world. Saemangeum Bay is located in the west of South Korean coast, see Figure 4- 4. To establish industrial complexes and develop farmland, a 33-km dike was constructed since 1991 and finished in 2010, closing the estuary area. The Saemangeum dike separated the bay from sea and reserves fresh water from the Mangyeong River(Lee, Lie, Song, Cho, & Lim, 2008). The big sea dike connects three islands and two land heads. It consists of four parts. The four parts have different height due to differences in bottom depth. The highest one is 35 m high and 290 m wide (base width).

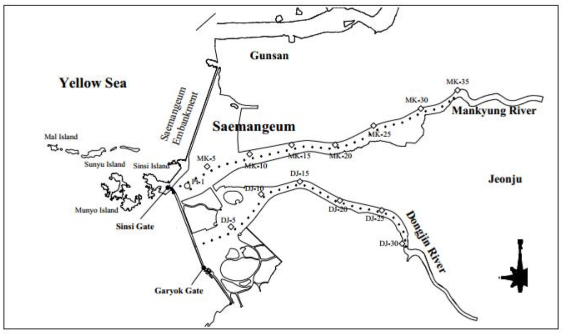


Figure 4- 4. Location of Saemangeum Dike

The project includes two sluices: Sinsi sluices and Garyeok Sluices (see Figure 4- 6). The sluices were constructed to discharge the flood flow from the inner river to the sea. The properties and dimensions of the two sluices are listed in Table 4- 2. Other gate types in the dike:

- Navigation: miter gate 4m * 13.85m
- Water level regulation and navigation: sliding gate 2.5m *2.5m
- Drainage: roller gate 6m * 2.5m

Table 4- 2. Summary Sinsi Sluice and Garyeok Sluice

Sluice	Sinsi Sluice	Garyeok Sluice
Gates number	20	16
Sea side	Radial gate: 30m width, 15m height	Radial gate: 30m width, 15m height
River side	Radial gate: 30m width, 12.5m height	Radial gate: 30m width, 12.5m height
Max. discharge	8,812 m3/s	7,050m3/s

Figure 4- 5. Left: location of Sluices; Right: Garyeok Sluices During Construction

Figure 4- 6. Left: : Sinsi sluices; Right: Garyeok Sluices

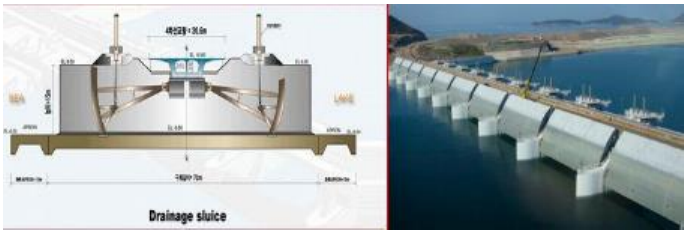


Figure 4- 7. Cross Section of Sinsi and Garyeok sluices

4.2 Proposed Barrier by Shanghai Municipal Government

The State Ministry of Water Resources and Electric Power claimed that Shanghai has to be protected floods with a return period of at least 1:1000 years. The Shanghai Municipal Government already initiated programs to upgrade the flood defenses, to keep the city safe against floods(J. Zhang, 2009). At present, the Shanghai Municipal Government has issued construction guidelines "*Project Planning Report on Storm Surge Barrier Construction in the Mouth of the Huangpu River (2002)*" and invents in flood defenses according to "*Criterion for Flood Control GB50201-94*".

According to the project planning research by the Shanghai Water Authority, 7 locations are put forward at the mouth, see Figure 4- 8.

Figure 4- 8. Proposed Barrier Locations in the Huangpu River (Source: Shanghai Water Authority)

The feasibility of the possible locations of a storm surge barrier is determined by various factors. Besides flood protection, navigation and construction are also of importance in such a densely populated area. There is a naval base at the mouth of the Huangpu River, which is a prohibitive factor for a barrier at this location. There are also lots of small ports and related industries along the Huangpu River. These cross sections are listed with short descriptions in Appendix C.

4.3 Conclusion

Three reference projects are studied to obtain knowledge and conceptual solutions for a design of a storm surge barrier in Shanghai. The abilities to apply in Shanghai are illustrated as follows:

1) Haringvlietdam consists of discharge sluices to drain water from the Rhine and Meuse River to the North Sea, meanwhile Shanghai needs this kind of sluices due to its dynamic water system (The system consists of Tai Lake, Yangtze River, Huangpu River and the East China Sea). They both require a large volume of water discharge capacity and located on soft soil foundation. Haringvlietdam is constructed on building pit, this construction method can be applied to Shanghai. However, the foundation treatment is kind of expensive and complex, more studies should be done to investigate cheaper solutions.
2) The Eastern Scheldt Storm Surge Barrier has an obvious advantage of minimizing the impacts on the ecosystem and maintaining salinity by opening sluices during normal

conditions. Also how to make such a heavy construction on delta soils can be used in Shanghai.

3) There are several similar points between the Saemangeum Bay and Shanghai area. They both locate in the estuary; they need barriers with the same functions, such as flood protection, water management and road connection. One problem is that Saemangeum is situated at the end of the estuary, however the Yangtze estuary end has a much more complicated system than that in South Korea, so it would be quite difficult to construct a barrier there.
4) All these three barrier systems are designed as a hybrid barrier system with different gate types in order to satisfy different barrier requirements. Which one or two would be the best solution(s) to apply to Shanghai needs more studies, it will be done in the further design phase.

5. DESIGN LEVEL I – FLOOD DEFENSE SYSTEM IN SHANGHAI

5.1 Introduction

Several solutions were proposed in this chapter for the purpose to strengthen the flood defense system of Shanghai, including raising floodwalls and constructing storm surge barrier(s). However, those options will have different ecological, environmental impacts, and the social impacts are extensive as well. The impacts can be positive or negative, depending on one's point of view, and partly depending on how the design implemented. This report is aimed to carry out a more detailed investigation of the alternatives. The impacts of all options are studied including ecological, economic and social impacts on Shanghai and surroundings. After that, the final selection is determined, according to a combination of cost estimation and Multi Criteria Analysis.

The water system in the study area is schematized in Figure 5- 1. The extreme flood can occur due to a combination of high water level in Huangpu River resulting from heavy rainfalls and typhoon induced storm surge at the same time. In that case, a closable flood gate is needed in upstream Huangpu River connecting the Tai Lake, and thus the extra discharge can be released to the Yangtze River. There are already several tidal gates constructed, so the influence from the Tai Lake is out of scope in this thesis. This study only focuses on the floods caused by typhoon induced storm surges.

The general requirements are set for the design of the protection system as follows.

- The hydraulic structures must provide a smooth and safe passage for all vessels passing through it.
- During normal conditions, navigation in both directions in the main channels should be possible.
- During the construction work, attention should be paid to the construction sequence in order not to hinder shipping.
- During normal conditions, the water flow from the upstream Yangtze River should be remained.
- Water salinity should be kept in the same actual conditions.
- The hydraulic structure should accommodate the complex and sensitive sediment environment in the Yangtze estuary.
- During normal conditions, the tidal movement should not be altered in order not to damage the local eco-system. The maximum reduction of tidal amplitudes after construction of hydraulic structures is set as 20% of its original shape.

- The projection is designed to block surge levels with a return period of 1/1,000 [1/year].
- The structure should be designed for a lifetime of 100 years.

A comparison between proposed alternatives of flood defense system are made, based on a study of the impacts on the local area and a rough cost estimation by using index numbers. The options must meet all the requirements listed above. The detailed descriptions of the alternatives are presented in the following section.

- Solution 0: Zero solution
- Solution 1: Reconstruction of urban floodwall and raising existing dikes
- Solution 2: One storm surge barrier at the mouth of the Huangpu River
- Solution 3: Storm surge barriers at the mouth of the Yangtze River
- Solution 4: Storm surge barriers behind the islands in the Yangtze River

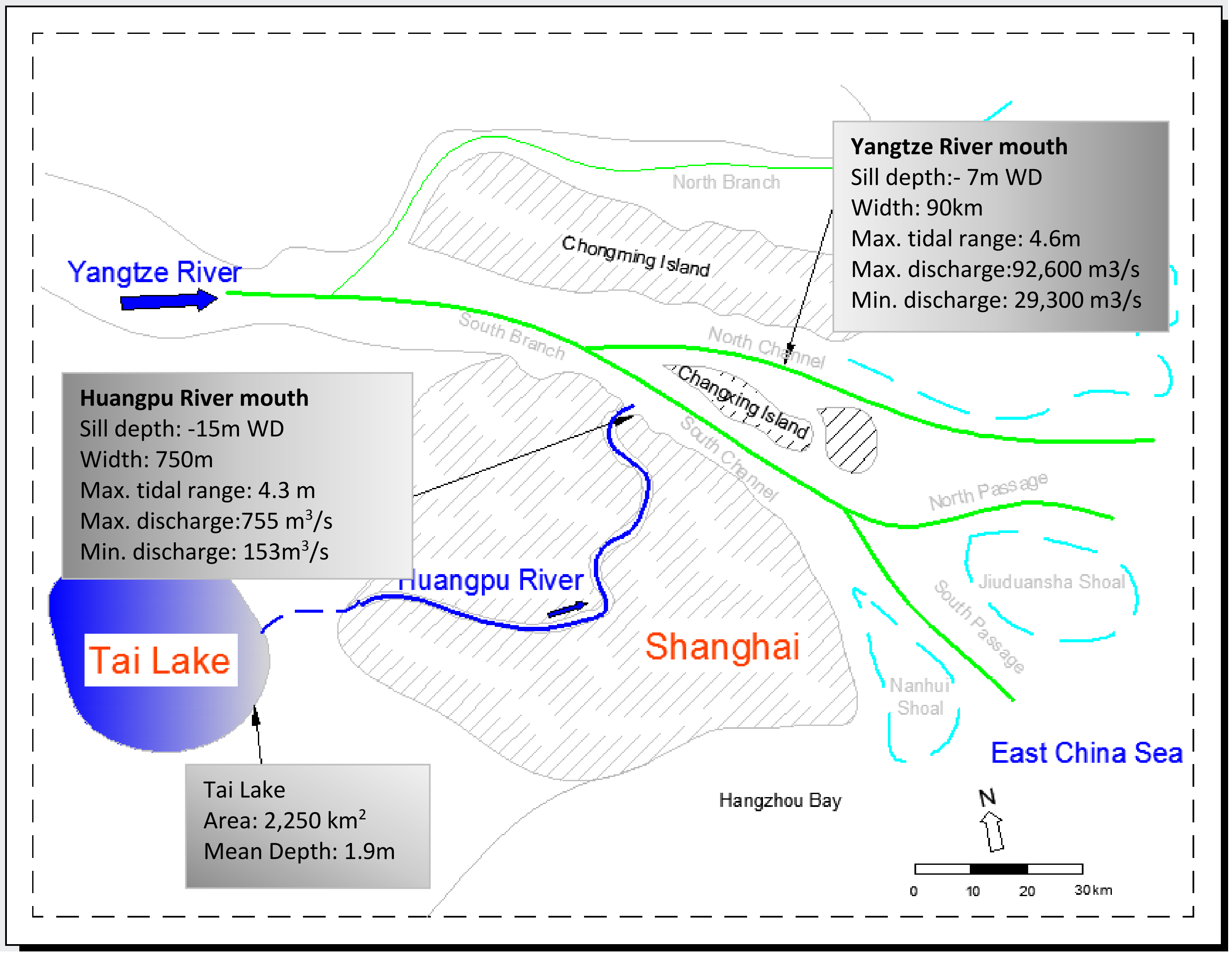


Figure 5- 1. Map of Schematized Study Area

5.2 Alternatives Description

Each solution is a combination of multiple defenses. The targeting protected area is the urban area of Shanghai, including the Chongming Island, which is located in the middle of the Yangtze Estuary.

5.2.1 Solution 0 - Zero solution

The solution 0 is the so-called "zero solution", which means doing nothing but keeping the present flood defense system as what is shown in Figure 5- 2. This system includes 315 km long sea dike, and 315 km long two-sided urban floodwalls, with an average top elevation of 6.9 m WD[4], to withstand high water levels(Nai, 2003). However, the flood protection of Shanghai, due to its unique location, is sensitive to sea level and land subsidence. The overall ground surface settlement is forecasted at 12 cm and the seal level will rise by 27 cm in the year 2050, according to the Shanghai Water Authority. Yin (2013) has already analyzed the tides in the past 30 years in Shanghai, they concluded the maximum tidal level would be 7.39 m in 2050(Yin & Xu, 2013). That means, almost the whole area of 6,340 km^2 will be inundated. Considering the significant economic status of Shanghai in China, the loss would be countless.

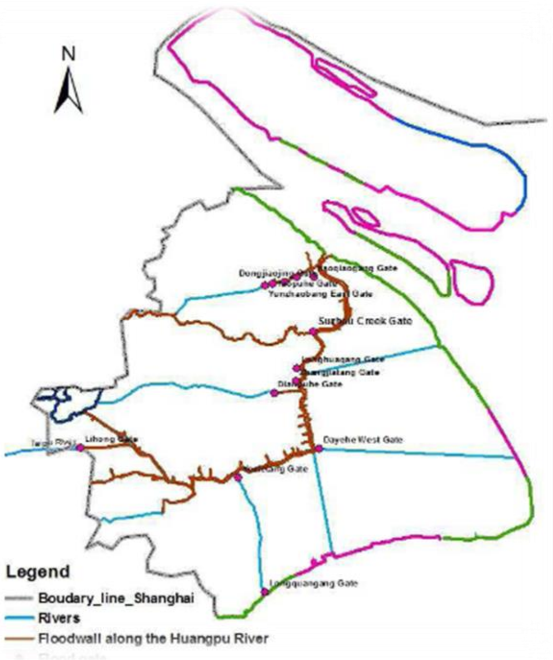


Figure 5- 2. Flood Defense System in Shanghai (Source: (Ke et al., 2013)), solution 0

[4] Which is 1.924 m lower mean sea level of China Yellow Sea; Hereafter, geographic elevation and water levels in the river and Sea are adopted at Wusong Datum.

5.2.2 Solution 1 - Reconstruction of urban floodwall and construction of new dikes

The solution one is to strengthen the floodwall and dike system. The urban floodwall must be reconstructed other than be raised, because of the poor current structures (yellow line in Figure 5- 3). The dikes, as a “dike ring” in the Netherlands (blue line in Figure 5- 3) around the island, which are 4 meters high on average at present, should be raised by 1.75m on average, so that the whole area (grey shaded area in Figure 5- 3) can be protected against the expected extreme storms. The design of the floodwall and dike raising is out of scope in this study. A typical cross section of dikes in the Netherlands can be adopted for a rough cost estimation in the following chapter, see Figure 5- 4.

Raising the existing double-sided 315 km long urban floodwall seems to be a simple solution. However, the concrete floodwall along the Huangpu River faces several problems that may be the unfavorable factors. To prevent the flood in the future, specifically 2050, the floodwall has to be raised by 1.5-2.0 m depending on different locations. In the last decades, the local government continuously constructed and heightened the dikes in order to meet the increasing demands for flood protection, which makes it nearly impossible to raise the poor floodwalls over again. The only way is to reconstruct the floodwall or build another line in the waterfronts. This can be quite expensive, difficult and less cost-efficient due to the densely population along the riverbanks. Construction of such long floodwalls could last for decades. In addition, 5 meters high floodwall can make Shanghai more risky of floods and it would block the beautiful scenery along the Huangpu River, which has been the “mother river” of the Shanghai city for 2,000 years.

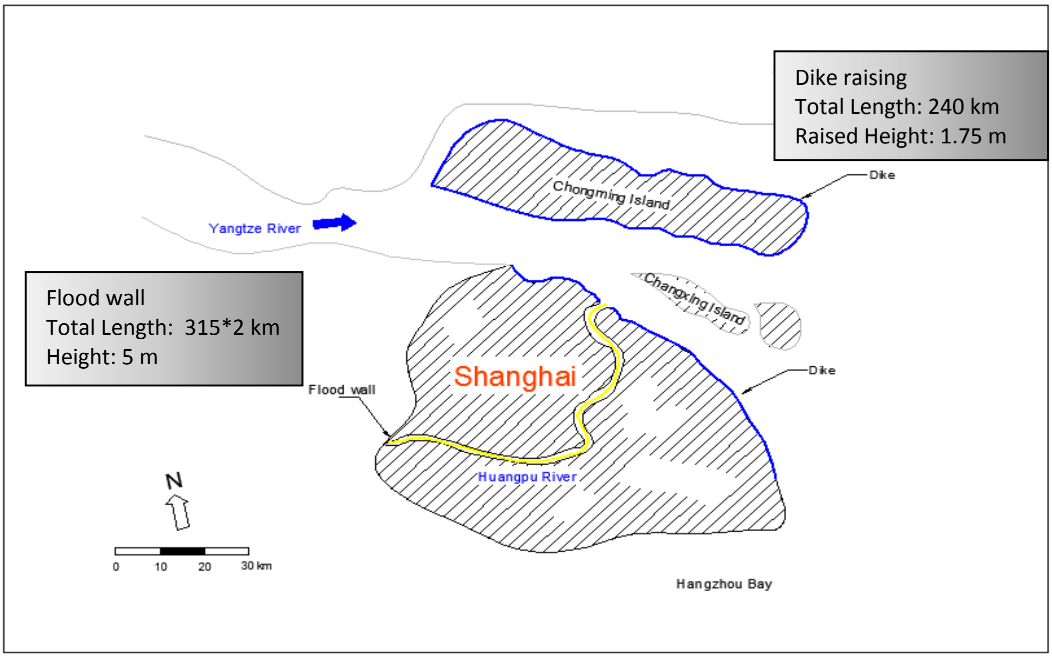


Figure 5- 3. Proposed Flood Defense System in Shanghai, Solution 1

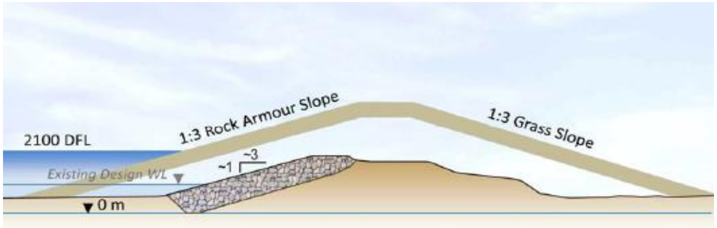


Figure 5- 4. Typical Cross Section of Sea Dike (Source: (Ministry of Environment of British Columbia, 2003))

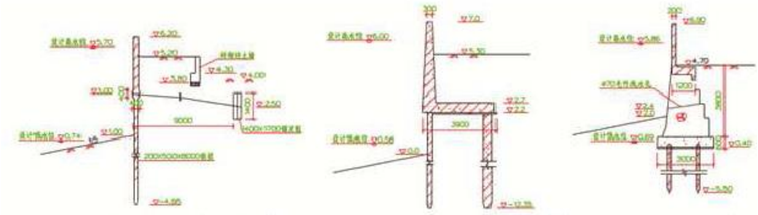


Figure 5- 5. Typical Floodwall Cross Section in Shanghai (Source: (Ke et al., 2013))

5.2.3 Solution 2 - One storm surge barrier at the mouth of the Huangpu River

This solution is to build a movable storm surge barrier in the Huangpu River. Generally, some problems will arise if the barrier is located more upstream. First, the protected area is reduced. Due to the denser population along the upstream, construction would be more difficult as less space is available. It is assumed the barrier would be constructed just at the mouth of the Huangpu River (red line in Figure 5- 6). According to a study by Yin (2013), it is found that the extreme storm flood elevation will rise to 7.17 m in 2030 and 7.39 m in 2050(Yin & Xu, 2013). The expected water head is roughly estimated to be 4 meters. Two additional dikes along the banks would have to be constructed along the south bank of the Yangtze River (blue lines in Figure 5- 6) to protect the urban area. Since the barrier would lead the water level to rise, the surrounding dikes (dashed blue line) should also be raised or reconstructed to withstand the raising water level. The island has to be surrounded by additional dikes.

When the expected storm surge is coming, the barrier will be closed. Since the average affecting time of a typhoon is around 12 hours, the barrier will be designed with an average closure time of 12 hours. However, the actual time may be longer if the barrier is to open when there is no head difference. In this case, the whole Shanghai area is under protection during floods. In more detail, the 14 million people living in the 4,944 km^2 urban area are protected with the barrier; meanwhile the other 700,000 residents from the island (1,215 km^2) are kept in safety by a dike ring.

However, looking back to history, the Huangpu River has been the most important river in Shanghai for thousands of years. Due to the importance of shipping in this area, river traffic must not be hindered by the barrier. And, being the most densely populated area in China makes it difficult for construction and thus limited space is available to store gates during normal conditions.

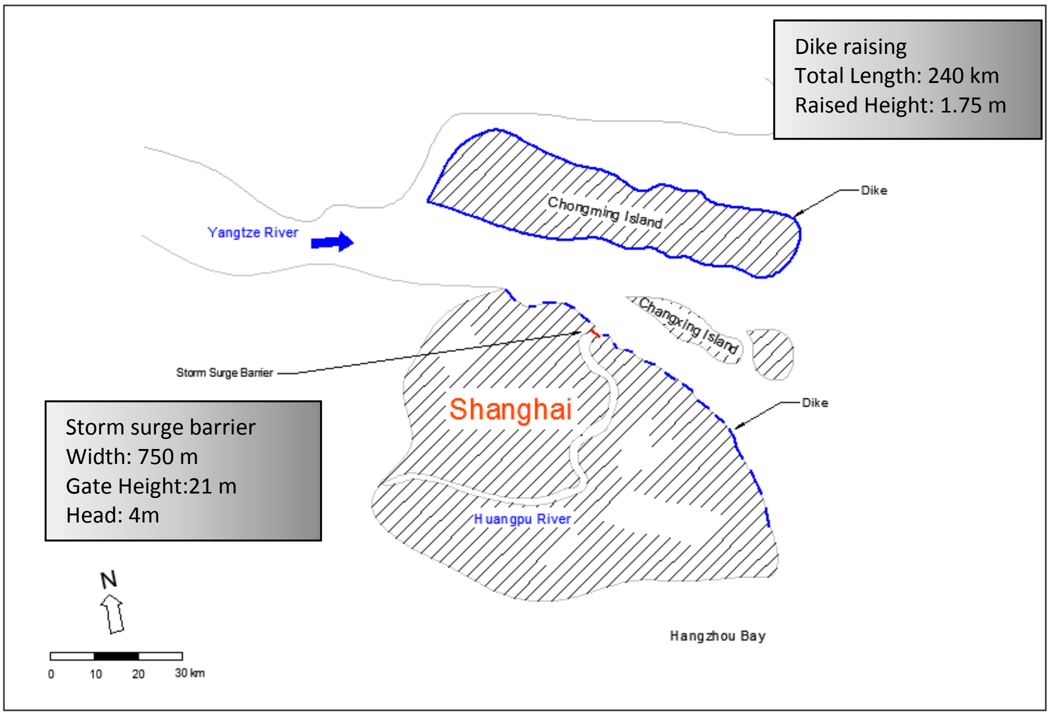
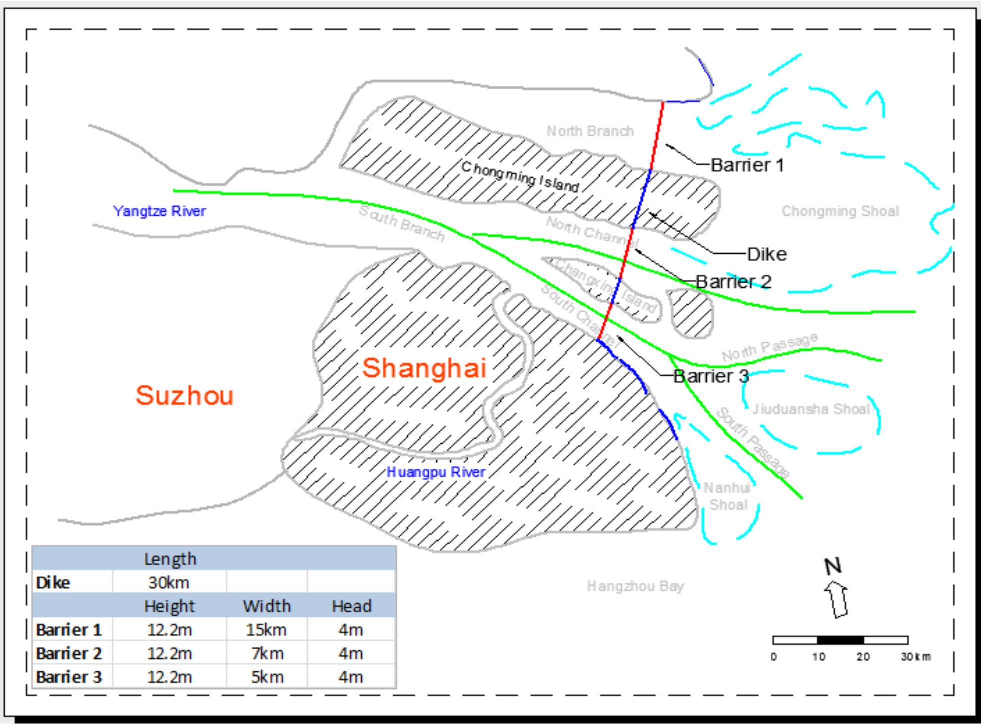


Figure 5- 6. Proposed Flood Defense System in Shanghai, Solution 2

5.2.4 Solution 3 - Storm surge barriers at the mouth of the Yangtze River

Another option is to close off the tidal inlets by constructing three storm surge barriers at the mouth of the Yangtze River, see Figure 5- 7. An obvious advantage of this alternative is, the barriers are in the more rural opening area, resulting in a much easier construction. The drawbacks are clearly seen, the complex sand-water dynamic system in this location makes the ecosystem quite sensitive. How to maintain the salinity and to please the nature should be taken into account. The shipping is also blocked during gate closure (the green line as the main navigation channel in the Yangtze River).

One additional benefit is, extra protected area can be gained including Suzhou. According to a flood simulation by Wang (2000), Suzhou, the major city of in Jiangsu Province, with a large population of 1.3 million, can be inundated under extreme floods induced by typhoon. As one of the richest cities in China, its development in the last 20 years is significant. The fast changing boundary conditions can result in a much more sever flood hazard to Suzhou. With the barriers blocking the tidal inlet, Suzhou can also be protected from flooding.

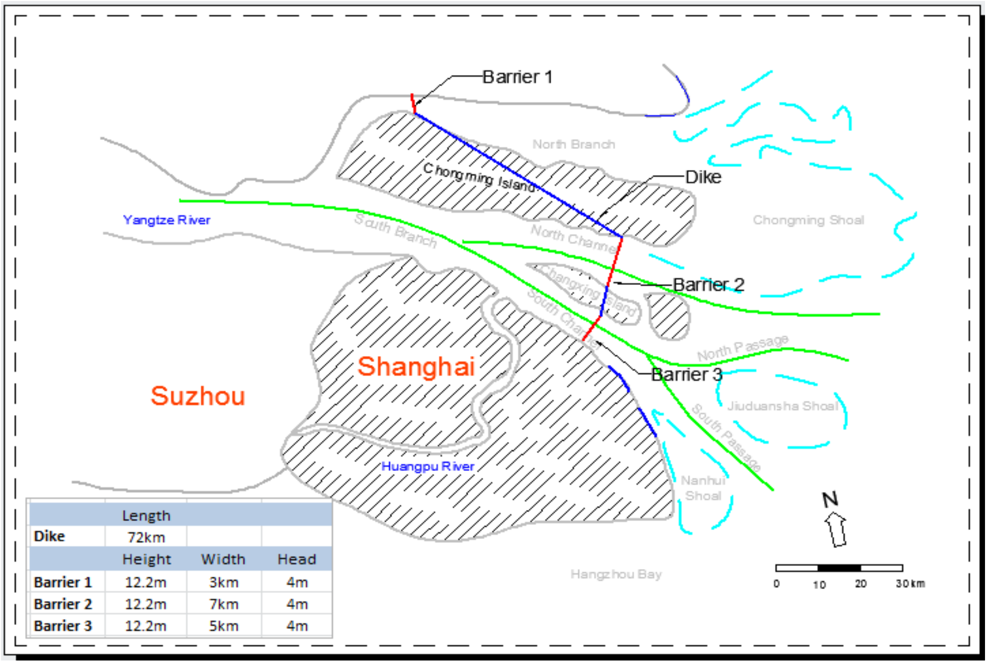


	Length		
Dike	30km		
	Height	Width	Head
Barrier 1	12.2m	15km	4m
Barrier 2	12.2m	7km	4m
Barrier 3	12.2m	5km	4m

Figure 5- 7. Proposed Flood Defense System in Shanghai, Solution 3

5.2.5 Solution 4 - Storm surge barriers behind the islands in the Yangtze River

The only difference between this alternative and solution 4 is the location of barrier 1. The construction cost of a barrier is relatively high, which is strongly related to the barrier width. So barrier 1 is moved more landward behind the islands, see Figure 5- 8.



	Length		
Dike	72km		
	Height	Width	Head
Barrier 1	12.2m	3km	4m
Barrier 2	12.2m	7km	4m
Barrier 3	12.2m	5km	4m

Figure 5- 8. Proposed Flood Defense System in Shanghai, Solution 4

5.3 Rough indication on costs

A (rough) cost estimation of the above four alternatives (except the alternative “zero solution”) is performed in this part by index numbers. The index numbers of a new barrier and dike raising are adopted from a report “Cost Estimation for a Canalized River Rhine (Waal)” written by Ad van der Toorn (2010)[5], floodwall calculation is performed according to a personal communication with to W.F. Molenaar (March 14th, 2014). The index numbers are modified in keeping with the actual situation in China, in order to be suitable to apply to Shanghai. Finally, sensitivity analysis is made by changing different index numbers.

5.3.1 Cost estimation

The cost of a new barrier or other hydraulic structures can be roughly estimated with the so-called index number/ unity price. The index numbers of construction, including barrier, dike and floodwall, are defined. The values are determined according to a report by Ad van der Toorn (2010) and a personal conversation with W.F. Molenaar (Personal Conversation, March 2014). More details refer to Appendix E.

Table 5- 1. Result Cost Estimation of Proposed Solutions

Alternative 1										
	factor	unit	length	unit					Total	unit
Dike	5,000,000	€/km	240	km					1,200,000,000	€
Floodwall	15,000	€/m	630,000	m					9,450,000,000	€
								Total	**10,650,000,000**	€
Alternative 2										
	factor		length	unit					Total	unit
Dike	5,000,000	€/km	240	km					1,200,000,000	€
	factor	unit	height	unit	width	unit	hea	unit	Total	unit
Barrier	15,918	€/m^3	21	m	750	m	4	m	1,002,834,000	€
								Total	2,202,834,000	€
Alternative 3										
	factor	unit	length	unit					Total	unit
Dike	5,000,000	€/km	30	km					150,000,000	€
	factor	unit	height	unit	width	unit	hea	unit	Total	unit
Barrier 1	15,918	€/m^3	12	m	15,000	m	4	m	11,651,976,000	€
Barrier 2	15,918	€/m^3	12	m	7,000	m	4	m	5,437,588,800	€
Barrier 3	15,918	€/m^3	12	m	5,000	m	4	m	3,883,992,000	€
								Total	**21,123,556,800**	€
Alternative 4										
	factor	unit	length	unit					Total	unit
Dike	5,000,000	€/km	59	km					259,000,000	€
	factor	unit	height	unit	width	unit	hea	unit	Total	unit
Barrier 1	15,918	€/m^3	12	m	3,000	m	4	m	2,330,395,200	€
Barrier 2	15,918	€/m^3	12	m	7,000	m	4	m	5,437,588,800	€
Barrier 3	15,918	€/m^3	12	m	5,000	m	4	m	3,883,992,000	€
								Total	**11,946,976,000**	€

[5] The Price is adjusted with 1% annual growth rate (inflation adjusted) considering the potential growth of Shanghai.

Sensitivity analysis

The above estimation is based on the index numbers, which are adopted by other existing projects. The price level in different places always has a huge variation. The cost is one of the main factors in determination of the design. As the exact current price is not available, a check base on a sensitivity analysis is implemented. The sensitivity analysis is performed by changing the unity price (see yellow cells in Table 5- 2).

Table 5- 2. Sensitivity Analysis of Index Numbers

	Index Numbers						
Dike	5,000,000	6,000,000	3,000,000	5,000,000	1,380,000	1,380,000	1,380,000
Barrier	15,918	15,918	15,918	30,000	10,000	15,918	15,918
Floodwall	15,000	15,000	15,000	15,000	15,000	10,000	30,000
Alternatives	Cost						
1	10,650,000,000	10,890,000,000	10,170,000,000	10,650,000,000	10,650,000,000	7,500,000,000	20,100,000,000
2	2,802,834,000	3,162,834,000	2,082,834,000	3,690,000,000	2,430,000,000	2,802,834,000	2,802,834,000
3	21,123,556,800	21,153,556,800	21,063,556,800	39,678,000,000	13,326,000,000	21,123,556,800	21,123,556,800
4	10,859,458,240	10,918,458,240	10,741,458,240	20,205,400,000	6,931,800,000	10,859,458,240	10,859,458,240

It is clearly shown that, the index number of dikes has quite small effects on the total cost of the whole system. The floodwall is only used in the first solution, so its influence can be ignored. Although the total cost is relatively sensitive to the index number of barrier, alternative 2 is always 3 to 5 times cheaper than alternative 4, whatever how the index number varies.

5.3.2 Loss of potential flooding disaster

Usually there is no direct benefit people can get from such public works, but only indirect benefits for the society concerning the extra safety against less risk of drowning or less economic damage. When people feel safety behind the flood defenses, and people and goods are easy to transport, the value of the construction is increasing.

The monetary goal for the entire system is to cost less than the projected benefits created by the Shanghai area. The initial construction cost of each solution is calculated in above section. The risk of a potential flooding equals the probability of a flood times the corresponding consequence. The design safety level of Shanghai is to ensure the flood defense system can block off a 1/1,000 year $^{-1}$ flood. Chen (2002) calculated the benefits due to protection from 1/1,000 [year^{-1}] flooding by evaluating the value of properties if they are inundated in different water depths, see fense system are well worth it.

Table 5- 3. According to Chen's result, if the flood caused the city inundated by 0.5 meters in 2050, the loss could amount to 30 billion euros. Looking back to the proposed solutions, even the most expensive one costs 21 billion euros. Compared to the losses due to the potential

floods, or in the other word, the benefit resulting from flood protection measures, the huge costs of flood defense system are well worth it.

Table 5- 3. Losses of Shanghai due to Floods (Resource:(Zhou, 2001))

Year	Possible Inundated Water Depth (m)			
	0.5	1	1.5	2
1999	€ 3,025,750,000	€ 3,610,625,000	€ 4,227,625,000	€ 4,909,625,000
2000	€ 3,177,000,000	€ 3,791,125,000	€ 4,439,000,000	€ 5,155,125,000
2010	€ 5,175,000,000	€ 6,175,375,000	€ 7,230,625,000	€ 8,397,125,000
2020	€ 8,429,500,000	€ 10,059,000,000	€ 11,778,000,000	€ 13,744,375,000
2030	€ 13,730,750,000	€ 16,385,000,000	€ 19,185,000,000	€ 22,388,125,000
2040	€ 20,324,875,000	€ 24,253,750,000	€ 28,398,500,000	€ 33,139,875,000
2050	€ 30,085,750,000	€ 35,901,500,000	€ 42,036,750,000	€ 49,055,125,000
2060	€ 44,534,250,000	€ 53,143,000,000	€ 62,224,625,000	€ 72,613,500,000

Table 5- 4 shows the approximate economic worth of the Shanghai area of the year 2013. This is approximately 36.25B Euros. Taking alternative 4 (cost of 10 billion euros) as an example for the cost benefit analysis in this part. Another big city Suzhou is protected in this case. That means, another 18B euros should be added. Since the initial cost of the proposed barrier project is 10B Euros, the project costs appear justified. Theoretically, this added flood protection would encourage investment in the Shanghai area, further increasing the barrier's benefits. And, considering the significance of the Port of Shanghai to China and to Asia, and the dramatic growing development of Shanghai, the costs appear even more justified. However, the construction cost of the entire flood defense system must be considered when comparing the costs with the future benefits. This especially will determine the overall design of the super dike ring.

Table 5- 4. Approximate Economic Worth of Shanghai Area of 2013 (Source:(Gu & Tang, 2002))

	¥ (Year: 2013)	€(¥/8)		People(2013)
China GDP	56.88 trillion	7.11 trillion	China population	1,300,000,000
GDP-per capita	39489	4936	Shanghai population	23,800,000
Shanghai GDP	290 billion	36.25 billion		

It can be concluded that, although there is a huge difference concerning the total costs between all those alternatives, even the most expensive one is well compensated for, due to the rapidly growing economy and economic status of Shanghai.

5.4 Multi Criteria Analysis

In this section, a comparison between previous defined alternatives is implemented. Multi Criteria Analysis is thought to be the best way to which option is the most favorable among a number of alternatives. In the real world, a large-scale investigation is performed among the different stakeholders, or a professional research is done by a group of experts. In this thesis, the multi criteria analysis (MCA) will be carried out by the virtual stakeholders.

The relevant criteria are identified and assigned a level of performance and relative impacts. The main stakeholders and their interest are described generally. The weighing factors of multi criteria analysis will be carried out according to interest of different stakeholders. The final scoring is done by two hydraulic engineers and one citizen from Shanghai.

The analyzed alternatives as proposed before, are summarized in

Table 5- 5.

Table 5- 5. Flood Defense System Alternatives Studied within Multi Criteria Analysis

Alternative	Flood Defense System Proposed
1	Reconstruction of urban floodwall and construction of new dikes
2	One storm surge barrier at the mouth of the Huangpu River
3	Storm surge barriers at the mouth of the Yangtze River
4	Storm surge barriers behind the islands in the Yangtze River

5.4.1 Stakeholders

For hydraulic structures, there are many different stakeholders. They may be direct users, supporting parties, public owners, environmental parties, etc. Each stakeholder has different requirements and interest. For this study, only the main stakeholders representing different parties are taken into account:

- The local government
- Ports
- Residents living/working in urban Shanghai area
- Residents living/working on Chongming Island
- Environmental specialists

In the actual situation, an extensive stakeholder analysis is needed to get the impression of the whole picture. In this first stage of design, only the general information, including the objectives, responsibilities and interest, is described in Table 5- 6.

Table 5- 6. Main Stakeholders and Their Interest

	Main Stakeholders	Description	Main Interest
1	The local government	The Shanghai Municipal Government plays a leading role in determining which project can be carried out in Shanghai. The main task for the government is to keep this city's rapid economic growth.	Financial benefits
2	Ports	There are quite a number of ports in Shanghai. At different points in the study area port activities are going on, port extensions are being planned. The detailed information of navigation requirements is stated in Chapter 7. There are two important port terminals in the study area. It is important that a downstream barrier will not have a significant influence on the handling time. The hydraulic structures in water will not hinder navigation and the delay due to construction should be avoid.	No delay of shipping
3	Residents living/working in urban Shanghai area	Shanghai urban area is the richest area in China. People living or working there are under the risk of being flooded.	Protection from flooding
4	Residents living/working on Chongming Island	The Chongming Island is the third largest island in China, which is known for its clean environment and fresh landscape. The remoteness makes the island separated from the development of chemical and industrial plants. This island is an important wildlife sanctuary with large area of farmland in China. In case the tidal characteristics and salinity are changed, a negative impact will be caused.	Unchanged natural tidal and salinity range
5	Environmental specialists	There are lots of environmental associations caring about the ecosystem in Shanghai. The construction will definitely cause pollution in land and in water.	Environment

5.4.1 Identification of criteria

A number of criteria are selected to analyze the benefits or drawbacks of each design alternatives. Every factor that is considered to be relevant for the decision-making process is listed below. The criteria are based on a check list by the British government (Gu & Tang, 2002) and a check list by PIANC (Labeur, 2007). The cost of each alternative is not taken into account.

Table 5- 7. Identification of Criteria Factors

Criteria			Description
Flood Safety	1	System failure	What would be the consequence if the system failed?
	2	Flexibility	Is it possible to upgrade the proposed system if the risks or safety level change in the future?
Construction	3	Materials	What is the availability of construction materials?
	4	Accessibility	Is it easy to access to the construction site?
	5	Impact on populated area	To what extend should the existing infrastructures be removed?
Ecological impacts	6	Pollution on populated area	Will the construction cause pollution (air, noise, light and soil pollution; excavation volumes)?
	7	Preservation of ecosystem	What is the impact on the present ecosystem, especially the wetlands?
	8	Salt water intrusion	Will the design help prevent salt water intrusion?
Economic impacts	9	Protected area&population	What is the protected area? How many people are protected?
	10	Disturbance of navigation	Will the navigation be blocked off or hindered during either operation or construction phase?
	11	Economic opportunities	Will it lead to additional economic development, like more job opportunities or a land mark for tourism?

5.4.2 Assignment of scores

The score range is set up in the following table, in order to describe in a simple and accurate way stating that the performance level of each solution can reach. After the definition of the range, each alternative gets a score for each criterion. The scoring is made from the stakeholders' perspective; then an average of the results is calculated.

Table 5- 8. Score Range of MCA

Expected Performance	Score
Ideal, regarding the analyzed factor	+4
Good, without significant problems	+3
Presenting some problems regarding the analyzed factor, but they can be solved	+2
Presenting significant problems regarding the analyzed factor, difficult to be solved	+1
Presenting challenging issues that could disallow the design	0

5.4.3 Assignment of weights

The weights of each factor are shown in Figure 5- 8. The detailed weighing assignments in perspective of the main stakeholders can be found in Appendix F.

Table 5- 9. Averaged Weighing Result of all Stakeholders

Criteria			Stakeholders 1	2	3	4	5	Average
Flood Safety	A	System failure	18%	18%	18%	16%	16%	17%
	B	Flexibility	11%	5%	5%	2%	4%	5%
Construction	C	Materials	2%	2%	2%	2%	5%	3%
	D	Accessibility	5%	5%	5%	9%	0%	5%
	E	Impact on populated area	9%	13%	13%	11%	9%	11%
Ecological impacts	F	Pollution on populated area	4%	15%	13%	9%	16%	11%
	G	Preservation of ecosystem	5%	9%	11%	13%	13%	10%
	H	Salt water intrusion	11%	4%	5%	5%	9%	7%
Economic impacts	I	Protected area&population	15%	9%	13%	9%	9%	11%
	J	Disturbance of navigation	15%	7%	5%	16%	9%	10%
	K	Economic opportunities	5%	13%	9%	7%	9%	9%
							Total	100%

5.4.4 Scoring result

The result of Multi Criteria Analysis is shown in Table 5- 10. Also the best two options of each criterion are colored in green. The detailed scoring assignments can be found in Appendix F.

Table 5- 10. Multi Criteria Analysis of all Proposed Alternatives

Criteria			Relative Weights(%)	Scores 1	2	3	4	Weighed Scores 1	2	3	4
Flood Safety	A	System failure	17.0	2.0	1.0	2.0	3.0	34.0	17.0	34.0	51.0
	B	Flexibility	5.5	1.0	1.0	2.0	4.0	5.5	5.5	10.9	21.9
Construction	C	Materials	2.5	2.0	3.0	2.0	2.0	5.1	7.6	5.1	5.1
	D	Accessibility	5.0	4.0	4.0	3.0	2.0	19.9	19.9	14.9	10.0
	E	Impact on populated area	11.0	2.0	2.0	3.0	4.0	21.9	21.9	32.9	43.9
Ecological impacts	F	Pollution on populated area	11.3	2.0	1.0	3.0	2.0	22.5	11.3	33.8	22.5
	G	Preservation of ecosystem	10.3	2.0	1.0	0.0	0.0	20.5	10.3	0.0	0.0
	H	Salt water intrusion	6.9	2.0	3.0	4.0	4.0	13.7	20.6	27.5	27.5
Economic impacts	I	Protected area&population	10.9	2.0	2.0	4.0	3.0	21.7	21.7	43.4	32.6
	J	Disturbance of navigation	10.4	2.0	2.0	3.0	3.0	20.8	20.8	31.2	31.2
	K	Economic opportunities	8.7	4.0	4.0	3.0	3.0	34.8	34.8	26.1	26.1
		TOTAL	100.0					**220.6**	**191.4**	**259.9**	**271.7**

5.5 Result

In the Multi Criteria Analysis (MCA), costs of each variant are not taken into account, because all criteria except costs are measures used to evaluate the performance. For some cases, it is possible to spend more money to enjoy a better performance. The weighed scoring and cost are identified in a rating/ cost graph as indicated in

Figure 5- 9. With the use of this rating/cost graph, a distinction can be concluded between the variants. Alternative 2 has the highest rating/cost ratio due to its much lower cost. Following is alternative 4, with the highest score but a little more cost.

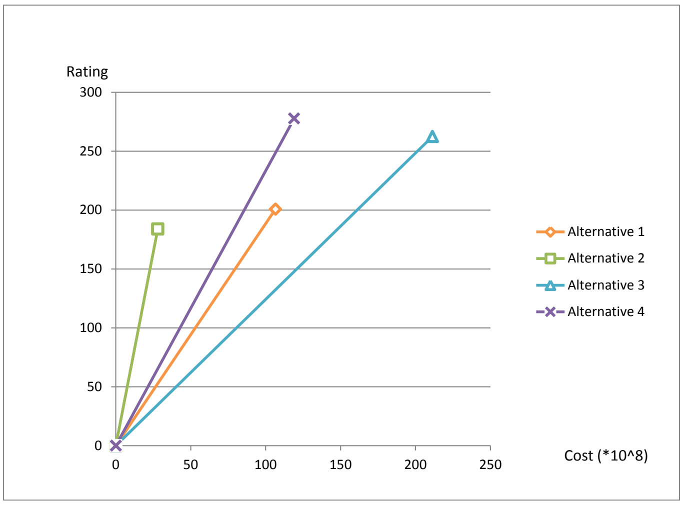


Figure 5- 9. MCA/ Cost Ratio of Each Variant

The final selection should be made between the best-ranked one (alternative 4) and the one of lowest cost (alternative 2). A summarized comparison between these two solutions is executed below in

Table 5- 1. It is easily understood that alternative 4 costs four times more than constructing a barrier in the smaller Huangpu River, because the cost is largely dependent on the dimensions of the barrier(s). Despite of the higher cost to build three barriers in the Yangtze River, the benefits are considerable.

Table 5- 11. Summarized Comparison Between Solution 2, 3 and 4

Alternatives	Cost (€)	MCA score	Protected Area	Protected Population	Vessels via Barrier Location per day	Construction Consideration	Other Aspects
One storm surge barrier at the mouth of the Huangpu River	2.8 billion	181	Shanghai ($6{,}159km^2$)	Shanghai (14 million)	1,000	- Quite difficult in greatly urban area - Large disturbance in populated area	- May help improve water quality in the Huangpu River - Higher flood risk due to much longer dikes
Storm surge barriers at the mouth of the Yangtze River	21.1 billion	290.7	Shanghai ($6{,}159km^2$) + Suzhou ($8{,}488km^2$)	Shanghai (14 million) + Suzhou (1.3million)	150	- Relatively easy construction in rural area - Disturbance on nature - Difficulty due to sedimentation	- Road connection between islands and surrounding cities
Storm surge barriers behind the islands in the Yangtze River	11.9 billion	275.8	Shanghai ($6{,}000km^2$) + Suzhou ($8{,}488km^2$)	Shanghai (14 million) + Suzhou (1.3million)	150	- Relatively easy construction in open rural area - Easy access to materials - Disturbance on nature - Difficulty due to sedimentation	- Help prevent salt water intrusion - Road connection between islands and surrounding cities

5.6 Conclusion

Four flood defense system alternatives were proposed in this chapter. All the possible solutions are intended to provide flood protection to Shanghai urban area as well as Chongming Island. A rough cost estimation with index numbers, as well as a Multi Criteria Analysis (MCA), is adopted as the evaluation method.

According to the cost estimation, the alternative of constructing one storm surge barrier at the mouth of the Huangpu River results to be the most cost-efficient at present. It is easily understanding that, this alternative is a consequence of current standards, which seems to be set by the local government, at this moment, for short and/or mid-term solution. When considering the future climate changes, the dikes have to be raised over or another defense line has to be built in the waterfronts. Including the potential extra cost, the final cost would likely double or even more.

The benefits due to flood protection are also taken into account. The estimated benefits in 2050 show that even the most expensive solution is well compensated. And, considering the significance of the Port of Shanghai to China and to Asia, and the dramatically growing development of this area, the costs appear even more justified.

In the Multi Criteria Analysis, the alternative of constructing three barriers behind the islands in the Yangtze River ranks the best for its excellent performance and huge additional benefits. The cost is not included in the Multi Criteria Analysis, for the reason that in some occasions it is reasonable to spend more money to realize a better performance. Except its main purpose as a flood protection structure, it also has multiple functions like prevention of salt-water intrusion, water quality improvement and road connection. In addition, the protected area (including Suzhou) and population are much larger. Furthermore, construction in the open rural area results to be a better choice, due to its less disturbance to the populated area. The problem of sedimentation in this location can also be technically solved.

To summarize, construction of three storm surge barriers behind the islands in the Yangtze River (alternative 4), with a better performance and lower cost, is the most flexible and cost-efficient one, and it guarantees a long-term safety to the whole region. For those reasons above, this solution is chosen for future study. The selected system is complicated. The design of surrounding dikes is out of scope in this thesis work.

6. BOUNDARY CONDITIONS

The part describes the boundary conditions for the selected storm surge barrier in the Yangtze Estuary. The design process must in satisfy these boundary conditions.

6.1 Hydrographic condition

For bathymetry maps of the Yangtze Estuary refer to Appendix G. Figure 6- 1 shows the depth profile of the select barrier system along the Yangtze Estuary.

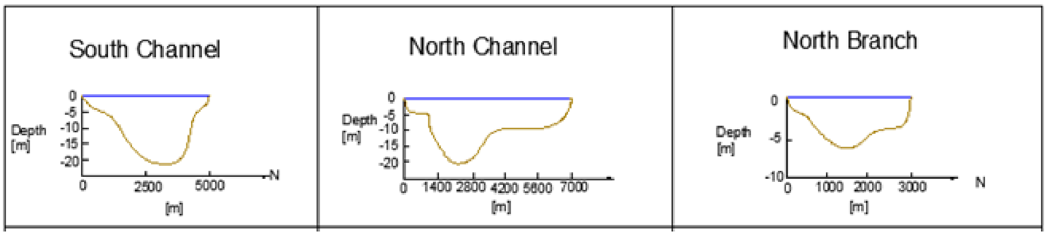


Figure 6- 1. Depth profiles of three channels in the Yangtze Estuary

6.2 Hydraulic conditions

These hydraulic conditions are discussed in normal and typhoon conditions separately. Non-storm conditions are relevant for barrier design in normal conditions. The future climate change (sea level rise), including the land subsidence is considered. Storm conditions concern the design return period of 1/1,000 [1/year]. From here on, when referred to the mean sea level, the level including 100-year relative sea level rise (0.5m+0.3m=0.8m) is used, unless otherwise stated.

Table 6- 1. Summary of hydraulic conditions

Non-storm conditions		
Tidal characteristics	Tidal range (mean)	2.7m
	Tidal range (maximum)	4.6m
Sea level rise	Absolute sea level rising height	0.5m
(for 100 year lifetime)	Land subsidence	0.3m
Discharge		
	River discharge (maximum)	92,600 m^3/s
Storm conditions		
	Maximum surge level	WD+ 6.3 m
1/1,000 [1/year] return period	Maximum waver height	5.5m
	Significant wave height	3.1m
	Peak wave period	12s

One important point to note is that, in the preliminary design, for the calculations of stability of structure, the wave conditions are present all the time. But this overestimation will result in a high overtopping discharge. It should be kept in mind, the stated boundary conditions could change with time. To make the design more cost-efficient, the time variation of the wave conditions should be taken into account. The use of maximum wave conditions is justified as it represents the most dangerous loading state. For check on stability of structure, information on variation thus is not needed. One pity is not enough hydrological data is available to perform a time-dependent design storm. The water levels during a design 1/1000 [1/year] storm can be regarded as the superposition of the astronomical tides and storm surges. Thus, an idealized design storm are chosen based on its peak surge levels, tidal conditions and storm period, see Appendix G.2.

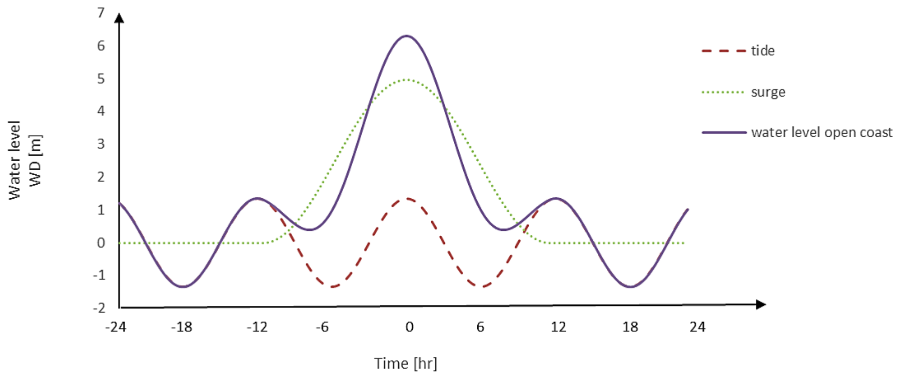


Figure 6- 2. Open estuary: water level for a 1/1,000 [1/year] design storm (data from Shanghai Water Authority)

6.3 Geotechnical conditions

Over thousands years of floods and river variation in the Yangtze Estuary, a complex and variable geology system is created. Layers of sand, soil, silt, clay are mixed with layers of organic deposits. The geological condition influences the engineering works. Unfortunately, the data on geology in the Yangtze Estuary is limited. Based on global research and reports, the geological cross section is composed. These data will be designed for the whole span along the Yangtze Estuary. In addition, the soil properties of the barrier location are not available at hand. The way is to assess it from the Dutch Standard. Of course these approaches are not correct, in order to get data in detail, field test as CPT (Cone Penetration Test) at the barrier location should be performed.

The soil conditions in the Yangtze Estuary are very poor; the upper layer of soil (down to -65m) consists of a loose to medium dense silty sand underlain by a layer of soft and compressible silty clay. Below the soft layer is the dense sandy deposit, which constitutes mainly the load bearing layers for the pile foundation. According to the *Chinese Code for Investigation of*

Geotechnical Engineering (GB 50021-2001), the soft soil in this location is classified in Figure 6- 2.

Table 6- 2. Schematized sub layers in the Yangtze Estuary (data from (Li, Chen, Zhang, Yang, & Fan, 2000))

Layer	Depth WD (-m)	Thickness D(m)	Class	Vertcally permeability (kPa)	Young's modulus Es (kN/m2)
K1	0~5	5	soft clay	1.0E-09	3000
K2	5~13	8	silty clay	5.0E-09	2500
K3	13~31	18	sily	5.0E-09	11000
K4	31~45	14	firm clay	1.0E-09	3000
K5	45~65	20	silty sand	8.0E-08	14000
K6	below 65		very dense sand		

6.4 Environmental conditions

The Yangtze Estuary is a large delta-front estuary with a huge amount of sediments coming from upstream river. The salinity gradient is located more at the front. Being a meso-tidal system, the Yangtze Estuary has a rich tidal ecosystem, with massive variety of tidal habitats and wetlands, such as sand flats, shallow open water and marshes. Water is the main factor in the management issue. Thus, special caution must be laid on construction of hydraulic structures in the estuary.

In a tidal basin constricted by barrier construction, during non-storm conditions, when the gates are all open, the flow area through the inlets are normally expected to reduce up to 40%-60% due to the constriction of river mouth. Also the tidal prism and tidal range are expected to reduce. Current velocities would be increased near the barrier and decreased in the estuary. The residence time of fresh water in the estuary is to increase and the salinity to decrease. Changes of hydrodynamics in the estuary can cause the loss of habitat and disturb the ecology. To migrate or prevent these effects, the tidal range reduction should be restricted to certain value caused by a constriction of the inlet.

6.5 Meteorological conditions

The Shanghai city enjoys a subtropical monsoon climate, with obvious features of continental climate. Most precipitation falls during the rainy season from July to September. Annual rainfalls average 1,227 mm.

Typhoons moving towards Shanghai not only bring high storm surges but also heavy rainfalls and strong winds to the area. These intense rains temporarily increase the runoff into Huangpu River substantially and may cause flooding of the Huangpu River when this occurs during barrier closure. The highest amount of rainfall Shanghai experienced was 200 mm in the aftermath of Typhoon Fitow in 2013, resulting just a small increase in water level inside the estuary.

7. DESIGN LEVEL 2 – BARRIER SYSTEM IN THE YANGTZE ESTUARY

7.1 Project area

Yangtze River Estuary is the part of the Yangtze River system downstream from Datong where the tidal limit is. The estuary has an approximate length of 630 km and a mouth of about 90 km width. The reach upstream from the junction (100 km from the estuary mouth) of the North Branch and the South Branch to Datong is called “the Upper Reach”, the downstream part is called “the Lower Reach” or “the River Mouth”. The limit of the tidal flood current lies near Jiangyin (240 km from the estuary mouth), while the tidal limit is located near Datong, the location of which can vary with the river discharge.

The Yangtze River Estuary is not a singer waterway to the sea. It is bifurcated into the North Branch and the South Branch. The North Branch is now suffering a severe sedimentation and the latter is the main passage for navigation and runoff. The South Branch is further divided by the Changxing Island, into the North Channel and the South Channel, and the latter is then again bifurcated into the South Passage and the North Passage.

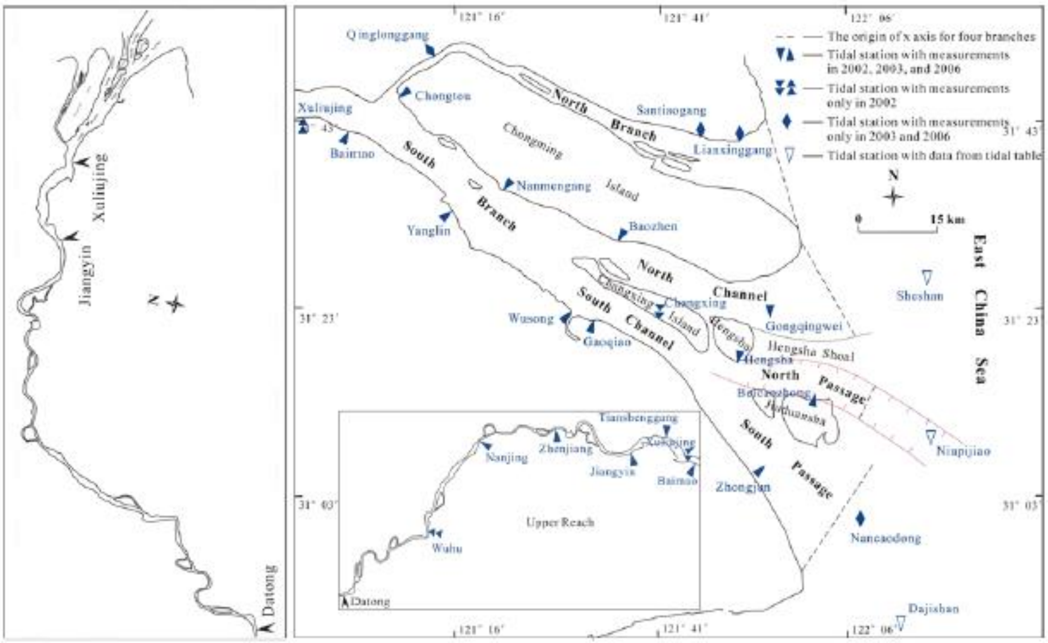


Figure 7- 1. Project area: Yangtze Estuary (Source:(EF Zhang, Savenije, Chen, & Mao, 2012))

7.2 Requirements for Yangtze River Estuary barrier design

This chapter describes the requirements for the selected barrier system in the Yangtze River Estuary. It includes the operational and functional requirements, particularly the nautical and environmental environments.

7.2.1 Operational concept description

In normal conditions, the barrier is open. The discharge of the Yangtze River flows freely to the sea. Shipping must be possible through the opened barrier.

When a high water level is expected, the barrier is closed. All navigation is blocked. The water level in the estuary should be kept under a safety level when the barrier is closed.

During the normal condition when the barrier is opened, the barrier must be able to allow water exchange between the Yangtze River estuary and the East China Sea to maintain the local eco-system.

7.2.2 Functional requirements

- Water retaining during storm surges
  - This storm surge barrier is part of an integral system to protect Shanghai against typhoon induced storm surges. Thus, the operational function should correspond to the whole flood defense system.
  - The barrier must be closed fast enough when a storm surge is expected.
  - During a storm surge, the barrier should be able to retain a high water level; the failure possibility should not exceed the pre-dominant standard.
  - During the barrier closure, there should be no river floods caused by accumulation of high water discharge due to hydraulic structures.

- Navigation
  - The hydraulic structures must provide a smooth and safe passage for all vessels passing through it.
  - During normal conditions, navigation in both directions in the main channels should be possible.
  - During the construction work, attention should be paid to the construction sequence in order not to hinder shipping.
  - There is a naval base behind the proposed barrier, in reality a special opening should be designed for the navy. However the detailed information on the navy is impossible to achieve, this aspect thus is not to be considered in this study.

- Passing of water
  - During normal conditions, the water flow from the upstream Yangtze River should remain.
  - The hydraulic structure must be able to discharge extra water from the retention area.
  - Water salinity should be kept in the same actual conditions.

- Discharge of sediment

- The hydraulic structure should accommodate the complex and sensitive sediment environment in the Yangtze estuary.
- It is expected the sediments can be partly deposited around the structure.

- Maintaining landscape
  - During normal conditions, the tidal movement should not be altered in order not to damage the local eco-system. The maximum reduction of tidal amplitudes after passing through the barrier is set as 20% of its original shape.
  - The hydraulic structure should meet the environmental requirements.

7.2.3 Operational requirements

Operational requirements are listed as following:

- Safety level

The barrier is designed to block surge levels with a return period of 1/1,000 [1/year].

- Lifetime

The structure is designed for a lifetime of 100 years. The main structural parts must be reliable for at least 100 years.

- Barrier closure duration

The storm surge barrier will protect the area from storm surges, but floods due to large river discharge accumulation during closure can still occur. When to close and how long the barrier can be closed can be an issue in design.

- Barrier opening duration

Opening of the barrier after a storm hits may take a few hours. The opening time should be limited to allow shipping as quickly as possible.

- Opening over water flow

After the storm leaves, the barrier will be open again. Because of the high discharge from Yangtze River, a high negative water head may occur. The barrier should be designed in that way that the structure is sufficient to sustain negative water heads. Also the bed protection should be considered because of potential erosion.

7.3 Division of sections

Except the barrier's fundamental function a flood defense structure, two main basic functions must be satisfied. One is to allow free shipping under normal conditions; the other one is to exchange water between the estuary and the sea so as to preserve the ecosystem. The barrier system is divided into two main functional sections: navigational section and environmental section[6].

More detailed calculations can be found Appendix H.

When analyzing the tidal dynamics in the Yangtze estuary, such assumptions are made as follows:

- The cross section of the estuary is generally assumed to be constant or gradually varying;
- The storage in the tidal flats adjacent to the channel, which affects tidal damping and the tidal wave celerity, is generally neglected;
- The friction term is generally linearized, with a constant friction coefficient along the channel;
- The three-order branches can be combined into a single channel, the estuary functions as an entity.

The barrier system in the Yangtze River estuary is schematized in Figure 7- 2:

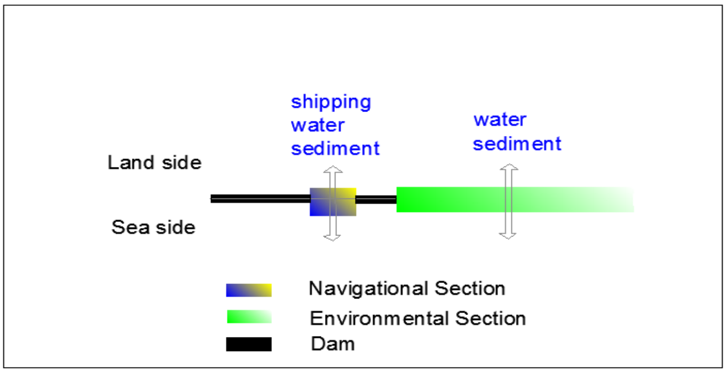


Figure 7- 2. Schematized barrier system in Yangtze Estuary as an entity

7.3.1 Navigational sections

The Yangtze River Estuary is the main waterway for both inland and coastal navigation in China. Therefore, the hydraulic structures should not, in any case, hinder the existing or potential navigation capabilities of the waterway. The structure should allow the largest vessel to travel through the navigation channel. In the estuary, this would be limited to the width and draft of the vessels to travel the narrowest and shallowest point along the vessel route. Thus the dimensions of the navigation channel should be chosen carefully, as well as the structure location. The erosion and sedimentation due to the construction of hydraulic structures would also restrict navigation, however, this aspect is out of scope in this study.

The navigational requirements are set according to a report “Approach Channel – A Guide for Design” by PIANC (1997) and “Waterway Guideline” released by Rijkswaterstaat (2011).

[6] Environmental section is defined as the section with extra opening of the barrier to allow water exchange.

- Vessel traffic characteristics

Before assessing the actual vessel traffic through the barrier location, the main vessel characteristics are discussed. By European standard, navigation requirements are completely defined by specifying the river class. PIANC (Permanent International Association of Navigation Congress) developed a worldwide used classification of vessels, including associated waterways.

It is important to note that the maximum clearance height of all the larger commercial vessels is 9.1 m. This clearance height can be adopted as the design criteria. The Shanghai Yangtze Bridge spanning the North Channel provides an averaged clearance height of 50 m, apparently quite sufficient for all vessels.

- Navigation channel dimensions

For the South Channel and the North Channel, both one-way and two-way navigation channel are considered. As the North Branch is seldom used for navigational waterway, only in some specific circumstances there would be some small fishery boats passing through the North Branch. In that case, a small opening is left for local people in need based on experience.

<u>Overview authorized waterways</u>

Yangtze Estuary is the main passage responsible for waterway transport in the Middle East China. Changjiang[7] Waterway Bureau and Shanghai Waterway Bureau have been working on the maintenance and improvement of the waterways for decades. The current authorized waterways are indicated in Figure 7- 3.

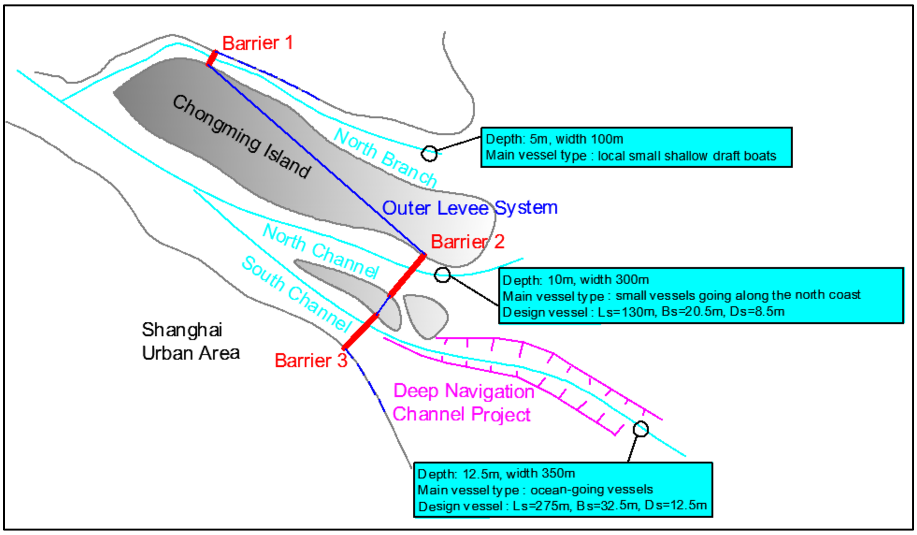


Figure 7- 3. Overview authorized waterways dimensions and design vessels

[7] Changjiang is the Chinese name of Yangtze.

With the type and dimensions of the design ships, the preliminary design of the navigation channel can be determined.

Table 7- 1. Authorized navigation waterways in the Yangtze Estuary

Channel Location	Channel type	Dimension of channel		Design Vessel					
		Width (m)	Depth (m)	Vessel type			Total length (m)	Beam Width (m)	Laden draft (m)
North Passage	Main navigation channel	350	12.5	ocean-going vessels	all weather conditions	4th generation container ship	275	32.2	12.5
					sailing with the tide	5th and 6th container ship	280	39.8	14
						100,000 DWT bulk ship	260	39	15.2
South Passage	Assistant navigation channel	250	8	smal vessels and shallow draft vessels going along the south coast	sailing with the tide	10,000 DWT bulk ship	130	20.5	8.5
North Channel	Assistant navigation channel	300	10	small vessels going along the north coast	sailing with the tide	10,000 DWT bulk ship	130	20.5	8.5
North Branch	under development	100	5	small shallow draft vessels among the nearby small cities					

Width Consideration

For the channel width design in straight sections, the bottom width w of the waterway, is given for one-way channel by:

$$W = W_{BM} + \sum_{i=1}^{n} W_i + W_B \qquad \textit{Equation 7- 1}$$

and for two-way channel by:

$$W = 2W_{BM} + 2\sum_{i=1}^{n} W_i + W_B + \sum W_p \qquad \textit{Equation 7- 2}$$

Where,

- W_{BM}, basic manoeuvring lane, for moderate channels, W_{BM}=1.5 B (design vessel beam width) , [m]
- W_i, additional width, [m]
- W_B, bank clearance, [m]
- W_p, passing distance, [m]

Depth Consideration

The navigation channel depth is estimated from:

- At rest draught of design vessel
- Tide height through transit of the channel
- Squat (vessel motion)
- Wave-induced motion
- A margin depending on type of bottom
- Water density and its effect on draught.

As the detailed information is absent, a simple approach is adopted. There is a minimum water depth required to enable ships to pass through the approach channel and lock. For maritime vessels a rule of thumb for the water of depth D_w would be:

$$D_w = D_S \times 1.15 + 0.5 \qquad \textit{Equation 7- 3}^{8}$$

Where D_S is the loaded draft of the design vessel.

The reference to design vessel is defined as:

- Beam B_S : distance between port side to starboard side
- Length L_S : distance between stern and bow at ships
- Draught D_S : distance between the undersides of the deck amidships to the keel's bottom

The minimum width and depth of the navigation channels can be calculated using the formula introduced by PIANC. The main dimensions are given in Figure 7- 4.

Table 7- 2. Required navigation channels dimensions

	Two-way channel		One- way channel	
	Width(m)	Depth(m)	Width(m)	Depth(m)
South Channel	172	16.25	88	16.25
North Channel	109	11	56	11
North Branch	60	5		

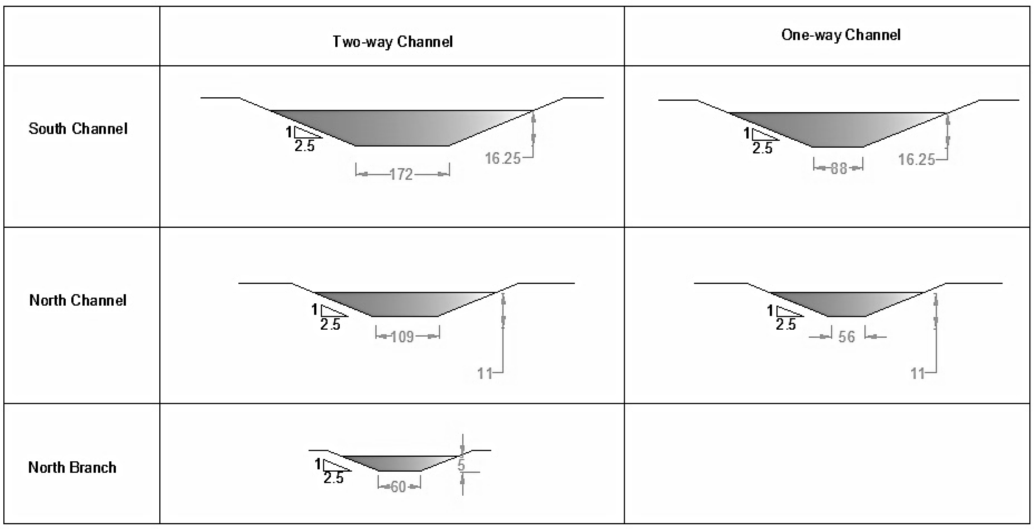


Figure 7- 4. Proposed navigation channels cross sections

[8] Note: this equation is limited for navigation of inland waterway design for the preliminary design. However, sailing ships via tides should be considered as the Yangtze Estuary is tide-dominated.

- Current velocity

One important to note is that, the prevailing longitudinal current velocity should not excess the allowed maximum velocity, which is defined as 1.5 m/s (3 knots) in normal conditions(PIANC & IMPA, 1997). According to the calculation result, the current velocities in all three channels under normal conditions are around 1.3 m/s, thus this requirement is satisfied as well.

7.3.2 Environmental sections

When considering the environmental aspect of the barrier, awareness in many people of the need to protect this area's natural resources and unique tidal habitat must be taken in to account. The reduction of the mouth of the estuary will cause a reduction of the tidal range. A main environmental requirement is set, as the barrier must allow the tides to enter freely, thus maintaining the tidal ecosystem.

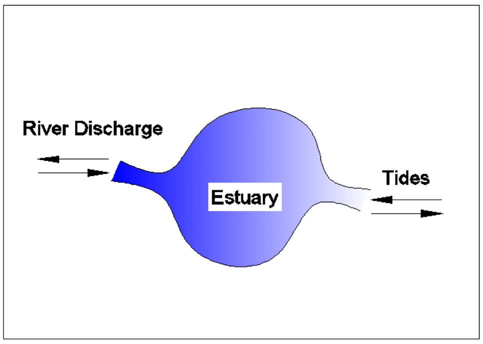


Figure 7- 5. Schematized water system in the Yangtze River Estuary

- Model set-up

A model is set up as a discrete system[9]. The basin is relatively small compared to the tidal wave length and surge, and it is partially closed except for a connection to the sea water. The assumption "small basin" can be applied here, in which situation the flow velocities in the estuary are quite low. Flow resistance and inertia are negligible. Therefore, the water level can be assumed to be horizontal all the times. The opening of the barrier system can be viewed as the inlet or gap, connecting the tidal sea to the basin. In this system, the only function of the basin is storage; its connection to the sea has an only function of transport. For detailed calculation refers to Appendix H.2.

However, according to this theory, the resistance in the estuary is neglected and the influence of the upstream river is not included. Considering the 8 km wide river mouth connecting the Yangtze River to the estuary, the high river discharge is not negligible. Assuming the influence

[9] This theory applies to such systems consisting of a nearly closed basin or a reservoir, connected through some narrow, short opening or a channel of some length to an external body of water with a time-varying water level.

from Yangtze River can extend to the tidal limit point as indicated in Figure 7- 6, an additional storage (shadow area in Figure 7- 7) is added to the basin.

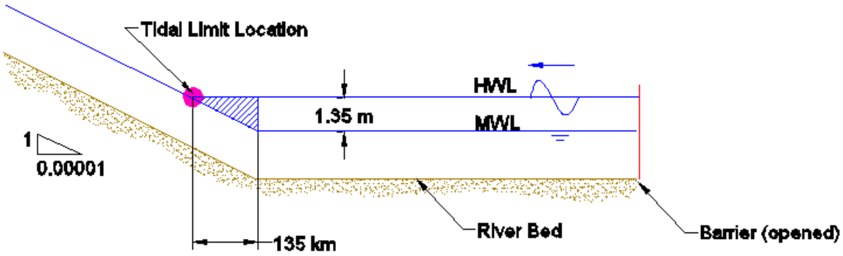


Figure 7- 6. Indication of idealized tidal limit in the Yangtze River

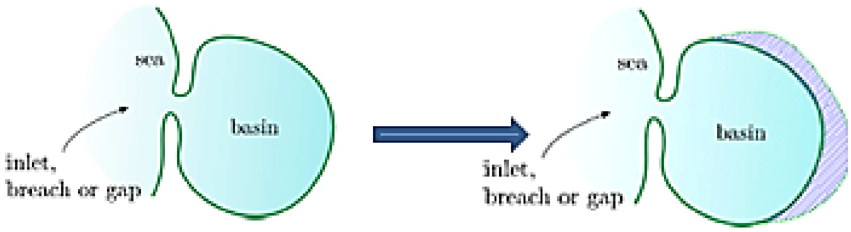


Figure 7- 7. Lift: original discrete system; Right: modified discrete system with additional storage area

The tides in the sea are semidiurnal; the tides can be characterized with a cosine shape:

$$\xi_s = \tilde{\xi_s} \, cos \, \omega t \qquad \text{Equation 7- 4}$$

Since the response is with the same frequency, it can be rewritten as

$$\xi_b = \tilde{\xi_b} \, cos(\omega t - \theta) = r\tilde{\xi_s} \, cos(\omega t - \theta) \qquad \text{Equation 7- 5}$$

Where,

- $\tilde{\xi_s}$ is the tidal amplitude in the sea [= 1.35 m]
- $\tilde{\xi_b}$ is the tidal amplitude in the basin [m]
- $r = \tilde{\xi_s}/\tilde{\xi_b}$, is the ration between the two amplitude [-]
- θ is the phase lag of the water level in the basin [rad]
- ω is the wave frequency $\omega = 2\pi/T$ [s^{-1})
- T is the wave period [~ 12h]

$$\Gamma = \frac{8}{3\pi}\chi(\frac{A_b}{A_c})^2 \frac{\omega^2\tilde{\xi_s}}{g} \qquad \text{Equation 7- 6}$$

Γ is the dimensionless parameter, containing all independent variables playing a role in the present problem [-]

- A_b is the surface area of the estuary [=1.8* 10 ^{9}m^2]
- A_c is the wet area of the opening [m^2]

- g is the gravitational acceleration [=9.8 m/s^2]
- χ is the dimensionless loss coefficient [=0.5]

Non-storm conditions

Under non-storm conditions, all gates are opened to allow water flow. Because the Yangtze River Estuary is a sensitive hydrodynamic system, but the habitats there are not that outstanding compared to the Scheldt Estuary in the Netherlands. The criteria is set as r=80%, referring to 87% in the design of Easter Scheldt barrier. In other word, after tides pass through the barrier, the tide amplitude is reduced to 80% of their original shape, due to the effects of the constriction of the river mouth. To satisfy this criteria, the minimum flow area is obtained as **A_c= 108,000 m^2**.

The maximum closure of the Yangtze River Estuary is calculated as

$$1 - \frac{108{,}000}{15000 \times 12.5} = 42.4\ \%$$ [10]

In a study by Ruijs (2011), the impact of a partial closure on the Galveston Bay's hydrodynamics has examined. He stated a maximum 40% reduction of the flow area is suitable for the design in the Bolivar Roads barrier. The situation of Bolivar Roads is similar to Yangtze Estuary. As a conservative estimate, a constriction of 40% is used in this thesis. In other word, 60% of the barrier system remains open during normal conditions.

Table 7- 3. Tidal response to partially closure

Situation	Total opening for water flow	Tidal response in Yangtze Estuary
Current	187,500 m^2	95% of incoming tide
60% closed	112,500 m^2	81% of incoming tide

The whole wet area of the environmental section[11]:

$$Y = 15km \times 12.5m \times 60\% - (X_1 + X_2 + X_3) = 105{,}616\ m^2$$

Assuming the average dredged depth over the whole span is 12.5 meters, the length of environmental section is around 8.5 km if the environmental barriers are fully opened.

Storm conditions

The whole opening during a storm can be composed of the length of the navigational sections, as well as the potential failure length of the environmental length. For a rough estimate, the

[10] The total span of the barrier system is 15km. The average depth at the location of the barrier is assumed to be 12.5m.

[11] X_1, X_2, X_3 are the wet area of the three navigation channels, they are 3,520 m^2, 3,001 m^2 and 363 m^2.

failure probability is adopted as 10%[12] during a storm. Thus, the whole opening length is: (172+187+60)+8500*10%= 1200m. This opening of 1200 meters will lead to a limited water level raise behind the barrier. The water level difference over the barrier (4 meters) results in a maximum flow velocity =$\sqrt{2gh} = 8.9\ m/s$, resulting an inflow of 135,000 m^3/s or 5.8 *10^9 m^3 during a 12h storm. Divided by the surface area 1800 km^2, this induces a 3.2m water level rise.

**Note: in the section, a modified discrete system is used for determination on required flow area. However, whether or not and how the river discharge would influence the theory is not yet known. Further research is recommended to be done with the help with more advanced software such as Delft 3D or Sobek making hydrological models. Due to limitation of the master thesis, only the analytical method is used as indicated above.*

7.3.3 Connection of main functional sections

Except the navigation section and environmental barriers there are additionally hydraulic structures to fully protection Shanghai. The levees can be either overtopping sufficient or resistant. It depends on how the barrier system is designed as stated in Chapter 8. The detailed design is not included in this thesis.

7.4 Influences of river flooding on landside areas

The obvious purpose of the barrier is to protect Shanghai and inner cities from coastal flooding due to storm surges. One of the potential disadvantages of the barrier is that it may cause flooding on the landward side of the barrier as the Yangtze River water levels are high or the barrier is closed for an extended period. River flooding occurs as a result of the accumulation of the high river discharge due to the obstruction of the continued river discharge by the barrier.

Then, a question arises, would the high river discharge with the barrier fully closed, in an extreme event, be sufficient to cause a freshwater flood behind (inland side) the barrier? To answer this question, the decision of if and when to close the barrier should be considered with caution in the design phase.

The decision of if and when to close the barrier is complicated in the case when the extreme surge and extreme discharge show up simultaneously. There has been a limited study on the joint probability of high Yangtze River discharge and storm surges. They are always treated independently at present, assuming the correlation of them is small.

This should not present a problem to the upstream cities, such as Nanjing, Changzhou and Suzhou, which are 200 km, 150 km, 50 km from Shanghai; respectively (see Figure 7- 8) The

[12] The gate failure is adopted from the design of the Eastern Scheldt Barrier. It is still sufficient to block the surge if approximately 10% of the gates fail to close during storm condition.

maximum duration of closure should be undertaken seriously. The problem can be solves in careful decision on two aspects: when and how long the barrier should be closed. When an extreme is expected, the barrier should be closed in advance. If the barrier is closed during low tides, there would be more storage area in the estuary to sustain the upstream water. Hence, the discussion is based on the type of flooding events and weather the barrier is closed at high/low tide:

- The surge tide coinciding with Yangtze River flooding (barrier closed at high tide)
- The surge tide coinciding with Yangtze River flooding (barrier closed at low tide)

Table 7- 4. Current situation of the Nanjing, Changzhou, Suzhou and Shanghai

	City	Population (million)	Distance from the Yangtze Estuary (km)	Max. Water level (WD)
B	Nanjing	6.5	200	11.2 m
C	Changzhou	3.5	100	11.0 m
D	Suzhou	1.3	50	11.0 m
A	Shanghai	14	0	11.5 m

Figure 7- 8. Location map of four cities in the lower Yangtze River

The results are shown in Appendix I. Obviously, the most dangerous event occurs when the Yangtze River at its maximum discharge and the extreme surge tide come together, which may draw coastal and river flooding at the same time. A starting point is to assume the barrier to be fully closed for 12 hours. During the closure, the estuary can be regarded as a large reservoir/ lake for water storage. The flow is assumed to be steady and uniform. Thus, the water profile along the upstream Yangtze can be estimated using the backwater curve formulae.

The water level in the estuary during the barrier closure becomes:

$$H_b = H_b, o + \frac{Q \times \Delta T}{A} \qquad \textit{Equation 7- 7}$$

Where,

- H_b, o: Original water level in the estuary before barrier closure [m]
- H_b: Water level in the estuary during barrier closure, [m]
- A: Surface area in the estuary [=881 km^2]
- ΔT: Barrier closure time [s]
- Q: Yangtze river discharge [m^3/s]

The backwater curve can be calculated using following equations:

$$h_i = h_e + (h_b - h_e)(\frac{1}{2})^{-\frac{x-x_0}{L_{1/2}}} \qquad \text{Equation 7- 8}$$

In which the boundary condition h=h_b at x=x_0 has been used and the so –called 'half- length' $L_{1/2}$ is given by:

$$L_{1/2} = \frac{0.24}{i_b}(\frac{h_b}{h_e})^{4/3} \qquad \text{Equation 7- 9}$$

Where,

- h_c: Critical depth, $h_c = (\frac{q^2}{g})^{1/3}$ [m]
- h_e: Equivalent depth. $h_e = (\frac{q^2}{i_b C^2})^{1/3}$ [m]
- C: Chezy coefficient, in index for bed roughness, for Yangtze River [C= 50]
- i_b: Bed slope [=1*10^{-5}]
- q: unit discharge, q=Q/B, B=8km

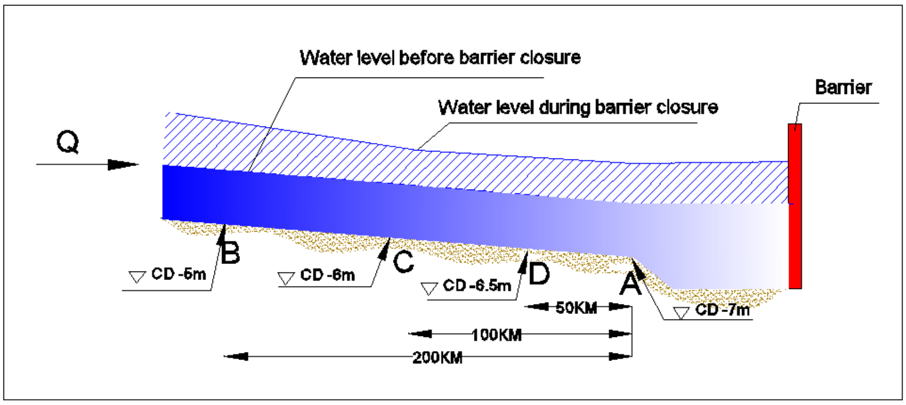


Figure 7- 9. Water Profile before and during barrier closure (not to scale)

The point in the surge cycle at which all the gates close will determine the water levels inside and outside of barrier. The barrier can be closed at high or low tides, resulting in different water levels in the reservoir. The results show, if the barrier is closed at high tide, the local levees along the Yangtze River in Nanjing (city B) cannot sustain such high water level, even though the closure period is reduced to 8 hours, which is already not sufficient to protect against the coastal flooding induced by storm surges, see Table 7- 5. In the opposite case, if the barrier is closed at low tide, which provides the estuary higher storage capacity, with the maximum discharge comes from the Yangtze River, those cities can still remain safe even the estuary is blocked off for 15 hours (see Table 7- 6).

Table 7- 5. Surface profile of Yangtze River when the barrier is closed at high tide for 8 hours

HIGH Q+HIGH TIDE, T=8H (closed at high tide)										
City	Q (m^3/s)	h0 (m)	T (hour)	he (m)	x (m)	L1/2 (m)	Depth (m)	Water level (WD m)	Allowed level (WD m)	SUFFICIENT
B	92600.00	17.33	8.00	17.50	200000.00	414477.80	17.26	12.26	11.20	N
C	92600.00	17.33	8.00	17.50	100000.00	414477.80	17.30	11.30	11.00	N
D	92600.00	17.33	8.00	17.50	50000.00	414477.80	17.31	10.81	11.00	N
A	92600.00	17.33	8.00	17.50	0.00	414477.80	17.33	10.33	11.50	Y

Table 7- 6. Surface profile of Yangtze River when the barrier is closed at low tide for 15 hours

HIGH Q+LOW TIDE, T=15H, (closed at low tide)											
City	Q (m^3/s)	q	h0 (m)	T (hour)	he (m)	x (m)	L1/2 (m)	Depth (m)	Water level (WD m)	Allowed level (WD m)	SUFFICIENT
B	92600.00	11.58	16.19	14.50	17.50	200000.00	378508.05	15.61	10.61	11.20	Y
C	92600.00	11.58	16.19	14.50	17.50	100000.00	378508.05	15.92	9.92	11.00	Y
D	92600.00	11.58	16.19	14.50	17.50	50000.00	378508.05	16.06	9.56	11.00	Y
A	92600.00	11.58	16.19	14.50	17.50	0.00	378508.05	16.19	9.19	11.50	Y

Note: The calculation is based on a simple analytical model, in which the flow is assumed to be steady and uniform. However, the flow condition is much more complicated as the Yangtze River has lots of bends and large amount of sedimentation transport in the river. The water profile should be estimated using hydrological modeling programs for a more accurate result, such as Delft 3D or Sobek 1D/2D.

7.5 Conceptual design

The paragraph above has elaborated how to determine different sections. This part describes the general idea on the whole barrier system in the Yangtze Estuary, including the locations and distribution of all sections. The proposed solutions are described in Figure 7- 10.

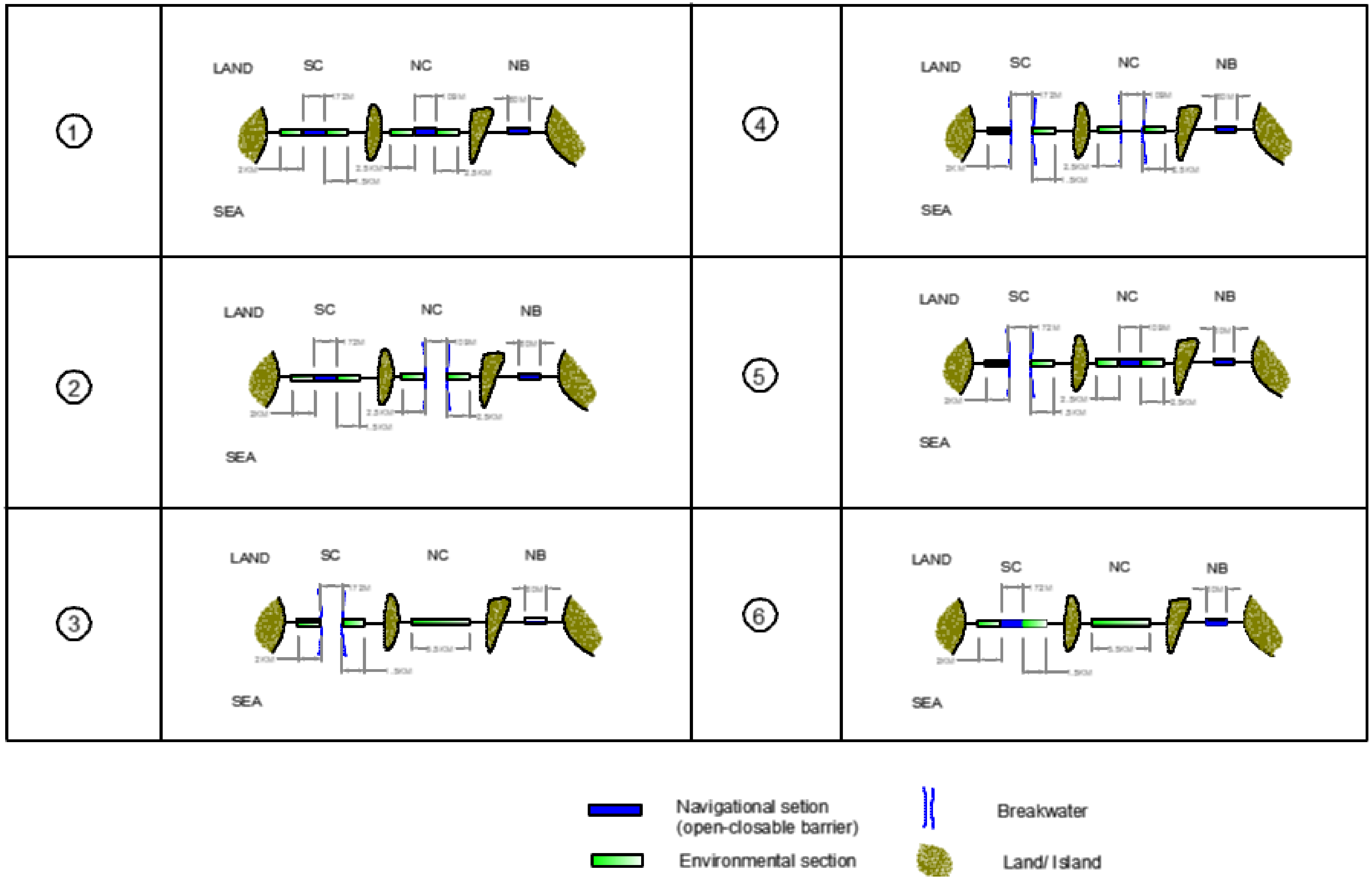


Figure 7- 10. Alternative barrier system in the Yangtze Estuary

7.5.1 Distribution of environmental sections

Before making the decision how the sections to be located, it is of importance to introduce the hydrological conditions of the three channels. Due to the perpendicular angle between the North Branch and the main channel, the fresh water inflow is very small: less than 1% of the Yangtze River discharge in recent years; during storm conditions the salty water intrusion can be sever in the North Branch(Erfeng Zhang, Savenije, Wu, Kong, & Zhu, 2011). Because of the quite small amount of water exchange, no extra opening (environmental section) is required in the North Branch. Only a small opening is left for shipping to fulfill demands from local fishery industry.

The North Channel and South Channel are responsible for almost all water exchange. The environmental section is divided in two parts according to the cross sections of the two channels. As stated above in section 7.3.2, the total length of the environmental section is 8.5 km. Thus, the environmental section in the North Channel is 8.5 km*(7/12) = 5 km, while it is 3.5 km long in the South Channel.

7.5.2 Open or closable navigational section

Several options are proposed for the barrier system in the Yangtze River Estuary, see Figure 7- 10. All the three channels, including the South Channel (SC), the North Channel (NC), and the North Branch (NB) are considered. The figures also refer to Appendix K.1.

In the North Branch, an open-closable barrier is to be built. One big advantage of this barrier is to decrease salt-water intrusion, as quite small amount of fresh water flowing from upstream.

In option 1, three barriers are designed for each channel with limited retaining height. However, cost is the main driver in design process. As the cost of a navigable barrier contributes to large part of total costs of the flood defense system, it would be wiser to reduce the number of navigable barriers. Because of the "*Deepwater Channel Regulation Project*[13]" in the South Channel, all vessels can be possibly shifted to pass through the South Channel, which means no navigation section constructed in the North Channel, referring to option 3&6. In this case a check on the travel density through the South Channel is performed, and no delay of shipping will occur.

Whether or not to close the navigational section during storm conditions is attractive to costs. In case of an open option, the environmental section must be able to sustain the storm surge. Thus, the water level and flow velocities inside the estuary should be checked during a storm,

[13] Deep Navigation Channel Project was completed in 2010, after when the South Channel became the main passage for all large vessels through the Yangtze River. The channel is maintained to 12.5 meters deep (referring to the mean low sea level) by regular dredging.

and a breakwater should be constructed at both sides along the navigational channel. Bed protection should be designed to increase bottom resistance.

Obviously, option 1 is the most conservative and expensive one, with three navigational barriers built in each channel. For option 4, the navigational channels in both SC and NC are kept open. Option 3 and 6 requires all ships to change shipping routes to SC. With construction of the real project "Deep Navigation Channel Project" in SC completed, it is possible to accommodate the vessels. Option 2 requires a navigational barrier in the SC, and NC to be opened, being just the opposite of option 5. Therefor option 3 and 6 are chosen for further calculation, named "open option" and "gate option" separately.

- Gate option (option 6)

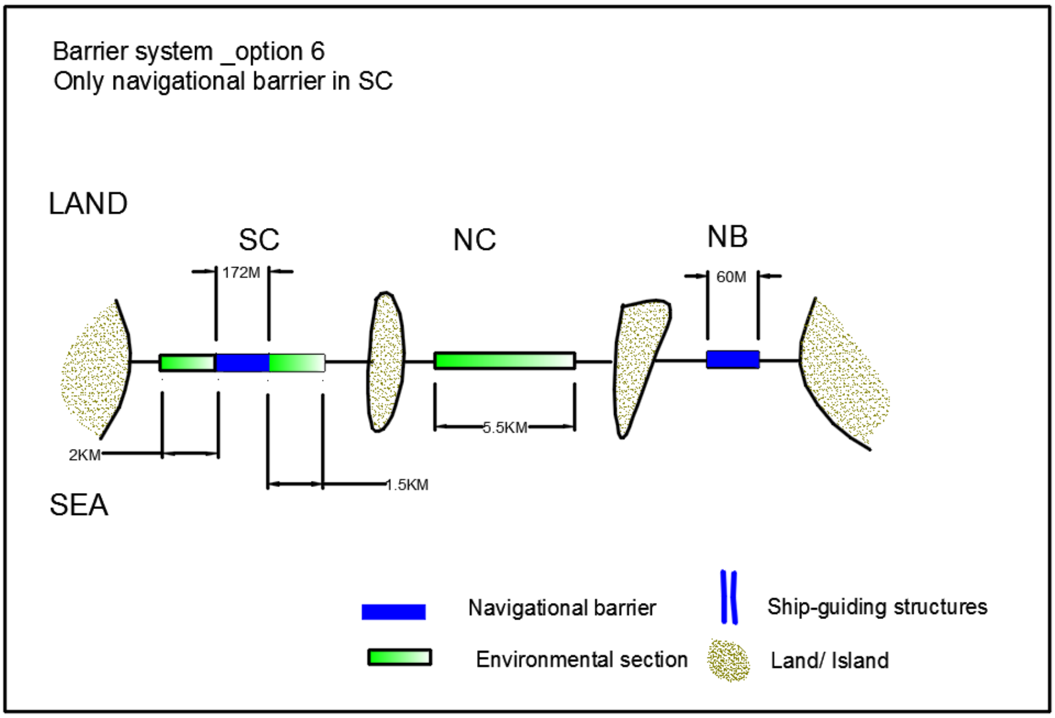


Figure 7- 11. . Schematic plan view of gate option

This option is a typical solution for flood defense system in the coastal areas. The calculation would lie on the determination of what the most cost-effective distribution of retaining height is. The detailed calculation will be performed in Section 8.2.

- Open option (option 3)

In this alternative, the navigational section of the barrier system is kept open as a navigational channel[14] allowing free shipping during normal conditions. The environmental section should

[14] For the current situation, the following is used: $L_{channel}$=1,500 m, $d_{channel}$=16.25m, $B_{channel}$=172m. The channel is protected by straight breakwaters at both sides. The bottom is protected with large stones ($D_{90=}$ 1m).

be able to fully block the surges. During storm conditions, floodwater can only flow through the channel.

This design seems to be cost-attractive as no extra navigable barrier is to be constructed. However, the water levels inside the estuary should be checked whether they stay within limits or not. Keeping such a narrow opening during storms would also cause large flow currents. Bed protection requires special treatment to increase bottom roughness and reduce surges.

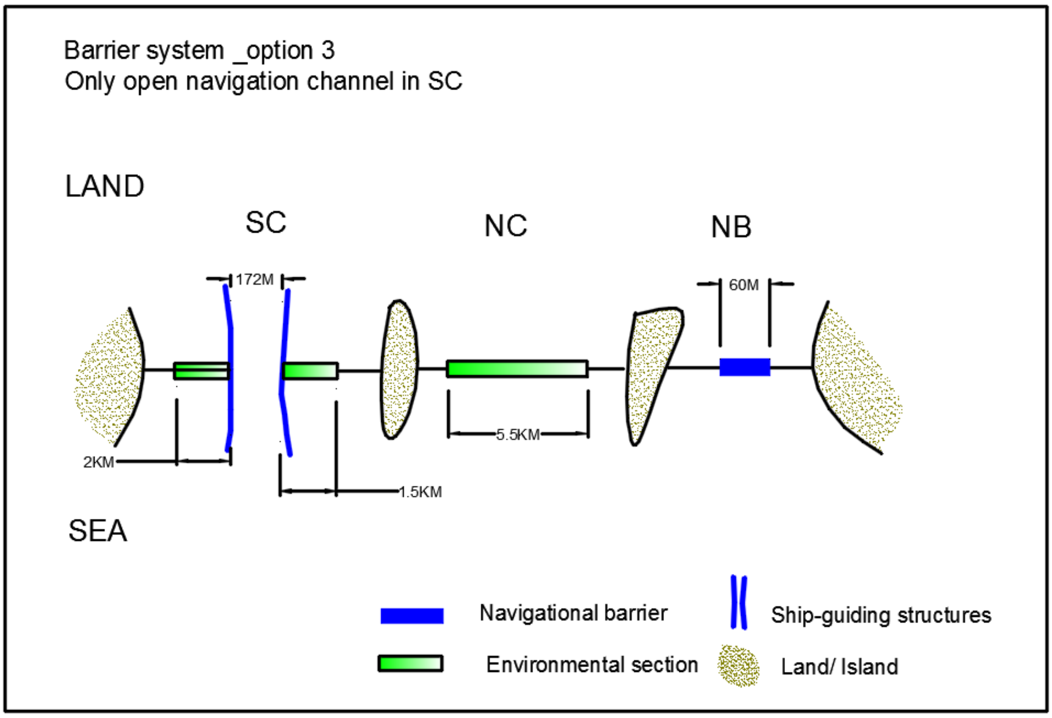


Figure 7- 12. Schematic plan view of open option

This system is schematized as a semi-closed basin. The channel length is relatively short compared to the surge length with the storage of the channel is negligible compared to the basin. "Rigid column approximation" is applied in the calculation(Labeur, 2007). Next the 1/1,000 [1/year] storm is released right in front of the barrier.

The result shows, the maximum water level is WD +3.1 m, which is a little bit smaller than the allowable water level WD + 3.5m[15]. Increasing the breakwater length leads to large resistance, which will result in lower water levels in the estuary. To ensure the safety behind the barrier, another calculation is performed using Math CAD, taking river discharge into account; and the results seem to be quite similar. Even one day after the occurrence of design storm surge, the water level inside the estuary is still 0.4m lower than WD + 3.5m.

[15] The maximum allowed surge level inside the estuary is WD+ 4.8m. A surge of WD+ 4.8 m occurs when the average water depth in the estuary increases to 16m (water level WD +3.5m), see Appendix J.

However, the current velocity close to the barrier can cause a problem to the bed protection, which may vary from -10m/s to +10m/s in this option. To solve this problem, one solution is to increase the channel width. Another advantage of bigger opening of the navigation channel is to reduce the length of the whole barrier, resulting in a lower cost. The calculation is performed with Math CAD (see Appendix K.2). The results show, with a 375 m wide open-navigation channel, the maximum water level inside the estuary arrives WD+3.5m when a design storm is expected. The current velocity then varies from -6 m/s to +8 m/s, see Figure 7- 13 and Figure 7- 14. In this situation, large differential negative head could occur, which should be kept in mind during barrier design process.

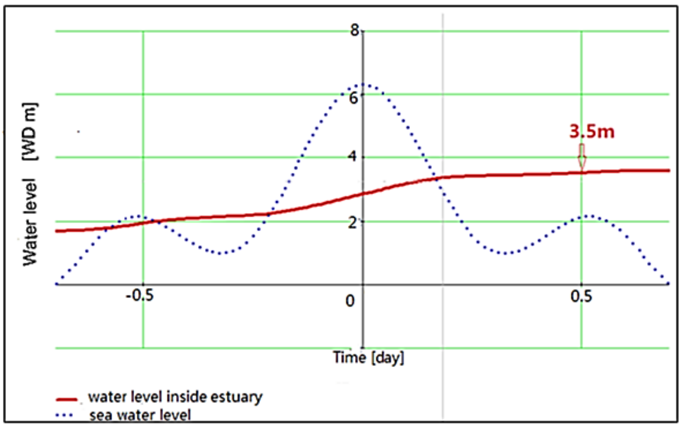


Figure 7- 13. Water levels inside the estuary in an open navigation section situation in design storm conditions (navigation channel width 375m)

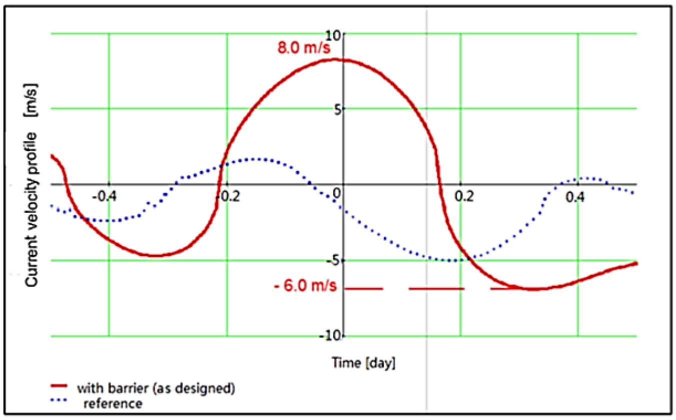


Figure 7- 14. Current velocity in the channel in open navigation channel option in design storm conditions (navigation channel width 375m)

7.6 Conclusion

This chapter describes the barrier design in the system level to get a conceptual design of the whole barrier system in the Yangtze Estuary.

The requirements and functional program are first set up for the barrier design. The barrier is divided into two mainly functional parts: the navigational part allowing free shipping, and the environmental part to maintain the ecosystem. The dimensions of navigational sections in three channels are calculated according to the design vessels on the channels separately. The criterion for determination on environmental section is to ensure the tides amplitudes remain 80% of its original shape after passing through the barrier.

The obvious propose of the barrier is to protect Shanghai and inner cities from coastal flooding due to storm surges. One of the potential disadvantages of the barrier is that it may cause flooding on the landward side of the barrier as the Yangtze River water levels are high or the barrier is closed for an extended period. Water levels in upstream cities are checked. If the barrier is closed at low tides, no flooding occurs when the design storm (1/1,000 year^{-1}) comes along with the large design river discharge, with a maximum barrier closure of 15 hours.

Whether or not to close the navigational section during storm conditions is attractive regarding to costs. Some alternatives of barrier system are proposed. The most attractive one is to keep the navigation channel open (no barrier) during storms. In case of an open option, the environmental section must be able to sustain the storm surge. Thus, the water levels and flow velocities inside the estuary are checked. Results shows, when the width of open navigation is designed as 375m, no flooding occurs inside the estuary. Applying a rough bottom protection helps reduce the flow velocities. As the high velocities (up to 8m/s) occur through the opening, the bottom protection stability should be further investigated and compared with cost. This 'open option' will be illustrated in more detail in Chapter 10.

In design level 2, two options are selected for further study. The report will go further with the gate option and open option. As the difference only stands on barrier in the South Channel, so design level 3 will only focus on the design of barrier in the South Channel.

8. DESIGN LEVEL 3 – BARRIER IN THE SOUTH CHANNEL

Because of the limitation of the Master's thesis, it is decided only to focus on the barrier design in the South Channel.

8.1 Barrier location and general site parameters

The proposed barrier in the South Channel is composed of a navigation channel and environmental barriers, which is already illustrated is Section 7.5. Due to the weak subsoil conditions, the foundation costs are expected to be decisive. This shortest path is chosen to the location of the barrier. The total span is 5km at this location, with 375 m navigation channel and 5km environmental barriers, as shown in Figure 8- 1.

The barrier is located at the head of Yangtze River mouth, connecting the Changxingxiang Island and the urban area of Shanghai. On the north bank, there is enough room to locate the barrier structures and/or auxiliary constructions if needed. On the south bank, the surrounding levees should be raised or strengthened in this entire barrier concept from a system point of view. As this thesis focuses on the flood protection structures, the levee system is assumed to have been strengthened to the required level.

Figure 8- 1. Project location of barrier system in the South Channel (not to scale)

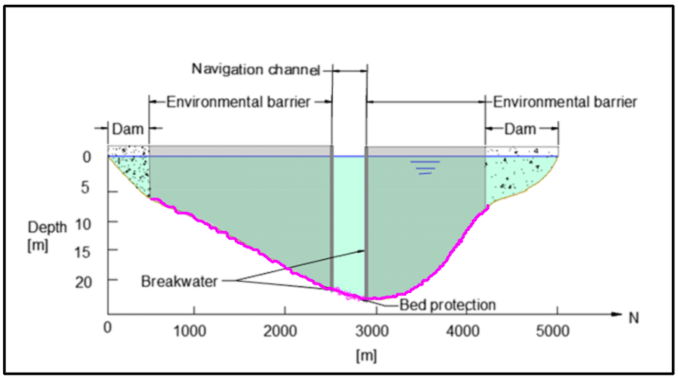


Figure 8- 2. Schematic front view of a barrier with open navigational section in the South Channel (not to scale)

8.2 Environmental section

This section presents an overview and evaluation of several alternatives for the environmental section. Most of the gate types presented in this part have already been constructed and have a great performance in the past decades. Additional information on these gate types refers to Appendix C.

8.2.1 Barrier alternatives determination

The first quick evaluation of different gate types focuses on four design criteria. Those criteria are equally weighed. They obviously do not cover all aspects and should not be equally weighed, but it is assumed this set suffices on this level of design. It is recommended to include more criteria and weigh their importance according to different stakeholders' interests.

The evaluation of different barrier gates focuses on:

<u>Structural aspects</u>

- Load concentration/ transfer of load to gate supports
- Sensitivity to hydrodynamic and wind load
- Span of the gate

<u>Operation aspects</u>

- Adaptability to negative differential head
- Closure in flowing water and waves
- Effects of silting
- Response for immediate operation when needed

<u>Flexibility and maintenance aspects</u>

- Flexibility for future changes

- Accessibility of inspection and maintenance
- Ease of replacement

Construction aspects

- Abutment and bed protection simplicity
- Construction method and experience with similar projects

To make a more detailed assessment of course it would be the best to take a preliminary draft for all alternatives. A good decision cannot be made just on simply comparing barriers on a few, equally weighed criteria. A detailed multi criteria analysis, along with a cost-benefit are recommended, as is done for flood protection system determination in Chapter 5. However this would be too much time consuming. Thus this barrier gate type selection would kept firefly. The scores on these criteria are specified by YES (+), NO (-), or MEDIUM (±). The result is shown in Table 8- 1.

Table 8- 1. Scores environmental barrier alternatives

		1	2	3	4	5	6	7	9
GATE TYPES		Vertical lifting gate	Flap gate	Swing barge gate	Floating sector gate	Segment gate	Radial gate	Visor gate	Inflatable rubble dam
CRITERIA	Structural aspects	±	+	+	±	±	±	-	-
	Operation aspects	+	-	±	±	±	±	+	±
	Flexibility and maintenance aspects	+	+	+	-	+	+	-	+
	Construction aspects	+	-	-	±	+	+	+	±

Two most important requirements help determine the barrier type. The first requirement concerns the hydrology of the Yangtze Estuary. The structure should be able to sustain negative differential head conditions at all design flow and wave conditions. Operation under flow and wave conditions are not feasible for sliding gates. This requirement also eliminates the flap gate and floating sector gate.

The other requirement is about siltation problem. The structure must be able to withstand some degree of siltation during high water discharge through the barrier. The prevention would minimize the maintenance cost and failure risk. This siltation requirement eliminates flap gate, swing barge gate and segment gate.

The flap gates, the vertical rotating gate, the floating sector gate and swing gate can be excluded because there are designed for special features, like allowing navigation and applicable to large spans which are not important for environmental sections. Instead, smaller spans are attractive because of their high repetition factor(Manual, 2007), due to the lower labor costs, machinery and equipment *etc*.

The inflatable rubber dam faces the durability issue because the rubber material. The fabrics may be replaced around every 30 years. Moreover, the rubber layer is quite sensitive to flow induced vibrations. But these disadvantages can be interpreted as an opportunity to the barrier more flexible to future climate change after 100 years.

The vertically lifting gate seems to have average higher scores than other gate types. A large span of the vertically lifting gates is feasible and gates are suited to reverse differential head and reverse flow during operation. But the gates in raised position are subjected to wind load. And the greater the water depth, the higher the gate then the higher the towers, due to this fact this kind of gate scores a "±" on its structural aspects.

8.2.2 Main structural dimensions

This section outlines the designed retaining height of the lifting gates (environmental section) and the associated incoming volume of water. In order to determine the retaining heights, two main requirements should be satisfied:

- Maximum water level rise inside the basin = 3.5 m
- Maximum current velocity close to the barrier as low as possible

Before the calculation is performed, some important assumptions are made as follows:

- The sea dike and levees around the Chongming Island have been heightened, and able to block the storm surges. They just allow a small amount of wave overtopping which will not increase the water level inside the estuary significantly. In other word, the only way for the storm surges travelling into the estuary is through where the barriers to be built.
- The relative sea level rise (0.8m) is accounted for by adding into water levels after computing the design storm as indicated in Section 6.2.
- The span of the navigational in the SC is 172m (determined in requirements), the environmental sections are 3.5km in the SC and 5km in the NC (determined in Section 7.5.1).
- The duration of the design storm is long enough that the water flow is spread out over the whole Yangtze Estuary.
- The storage area of the Yangtze Estuary is 1,800 km^2 (determined in Section 7.3.2).

- Full retaining lifting gates

In the option of keeping the navigational section permanently open during all conditions, the environmental section is assumed to be fully retaining. In other word, flow only passes through the open navigation channel. Section 7.5.2 has illustrated that if then open channel is decided to be 375 m wide, 16.25 m deep, the maximum allowed water level rise behind the barrier is 3.5 m, but the current velocities (varying from 8m/s to -6 m/s) are dramatically larger than normal.

Estimated structural dimensions of the lifting gates are given below.

Table 8- 2. Estimated structural dimensions of the lifting gates in the South Channel (fully retaining)

Parameter	Value	Remarks
Opening width	80 m	Experiences from reference projects shows that using gates with width of 75-100 m provides the most cost-effective solution
Number of openings	42	Based on the gate width of 80m, required total section width of 3.2 km
Top level closed gate	WD +6.8 m to WD+ 8.1m	Maximum design surge level WD+ 6.3 m. Freeboard is in the range of 0.5 to 2x the wind wave height (0.9m) to reduce surge and keep gates visible during surge events.
Sill level	WD -12.5 on average	Based on requirements for environment
Total height of gate	19.5 m	Based on a top level of WD +7m

- Limited retaining

Another option is to have both sections designed with open-closable gates. The cost of a barrier is mainly dependent on the main dimensions, including the maximum water level difference above the barrier, the height of the retaining construction and the barrier span. The influence of construction height (retaining height) can be minimized by optimizing the retaining heights for different sections. The retaining height for environmental section and navigational sections need not to be the same level. Peter A.L. de Vries (2014) stated, for the Bolivar Roads reduction barrier an equal retaining height over the full whole span is the least costly. The conditions in the Bolivar Roads are assumed to be similar to the system barrier in the Yangtze Estuary, thus the following calculation is based on that both the navigational section and environmental section have the same retaining height.

In this option, overtopping and/or overflow over the gates and levees occur. The barrier is modeled as a (submerged) sharp crest weir. In order to prevent large force on the backside of the structure, the maximum combined discharge over the closed gates is set at 10 l/s/m. A cross section of the gate, presenting the maximum hydraulic conditions is presented in Figure 8- 3.

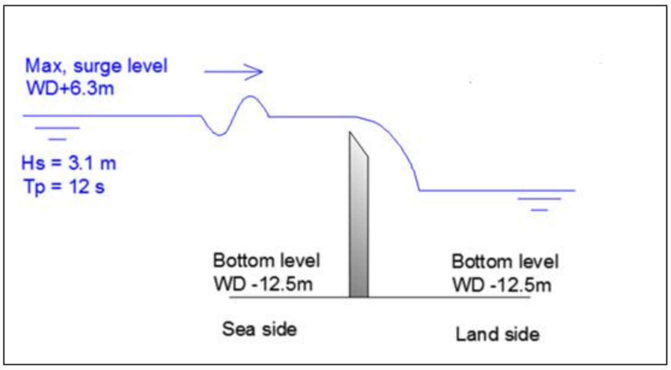


Figure 8- 3. Modeled lifting gate and designed conditions

The maximum allowable water level rise at the retention area at the maximum surge event can now calculated according to *EuroTop Manual: Wave Overtopping of Sea Defences and Related Structures (2007)*, by combining the following criteria:

- As overtopping discharge of 10 l/s/m is the maximum allowed over the gates;
- The incline of the current primary levee is set at 1:1.5, the crest height at WD +5m;
- The significant wave height is assumed under non-breaking conditions (breaker index>2).

Appendix K.3 presents the detailed calculation. It follows the maximum water level rise allowed in the retention area due to overtopping and/or overflow of the primary levee is 3.5m. First, the crest height of the primary levee is determined.

Average overtopping discharge (EuroTop Mannual, 2007)

$$\frac{q}{\sqrt{gH_{m0}^3}} = \frac{0.067}{\sqrt{tan\alpha}}\gamma_b\xi_{m-1,0}exp(-4.75\frac{R_C}{\xi_{m-1,0}H_{m0}\gamma_b\gamma_f\gamma_\beta\gamma_v}) \qquad \textit{Equation 8- 1}$$

With a maximum of:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.2\ exp(-2.6\frac{R_C}{H_{m0}\gamma_f\gamma_\beta}) \qquad \textit{Equation 8- 2}$$

Where,

- Tanα= angle outer slope of the seaside of the levee = 1:1.5 [-]
- Rc= retaining height of levees [m]
- γ_β = influence factor for angled wave attack = 1.0 [-]
- γf = influence factor for roughness elements on slope = 0.7 [-]
- γb = influence factor for a berm = 1.0 [-]
- ξ= break index = 2.8 [-] (non-breaking)
- H_{m0}= significant waver height = 3.1 [m]

It shows that under the leading wave conditions, the formula for non-breaking waves should be used. Give a perpendicular wave attack, the minimum required retaining height of the levees under given conditions is equal to 8 m. The corresponding water level rise due to overtopping of the levees is 1.5 m.

The maximum allowed water level rise due to overtopping of levees and gates are 3.5 m. This results in an allowed water level rise due to overtopping and/or overflow of gates of 3.5-1.5 =2.0 m. This value is used as input for the iterative calculation of the minimum required gate height. The result presents the minimum crest height of the gates, concerning the allowed water level rise in the retention are is WD +5.1m, with an overtopping discharge of 9.49 l/s/m.

Table 8- 3. Estimated structural dimensions of the lifting gates (limited retaining)

Parameter	Value	Remarks
Opening width	80 m	Experiences from reference projects shows that using gates with width of 75-100 m provides the most cost-effective solution
Number of openings	44	Based on the gate width of 80m, required total section width of 3.5 km
Top level closed gate	WD +5.1 m	Based on the maximum overtopping and overflow requirement
Sill level	WD -12.5 on average	Based on requirements for environment
Total height of gate	17.6 m	Based on a top level of WD +5.1m

8.3 Navigational section

As stated above, the navigational section is kept open as a navigation channel during all circumstances. As the real project “Deep Navigation Channel Project” conducted in the South Channel, all vessels used to pass through both the South Channel and the North Channel are forced to travel this navigation channel in the South Channel. A check on the traffic intensity has been performed that no delay of shipping will occur. The channel is located 2,500 m south to the Changxingxiang Island, thus no modification of the entrance channel is required.

The dimensions of the navigation channel are calculated in Section 7.3.1. According to Section 7.5.2, when the width of open navigation is designed as 375m, no flooding occurs inside the estuary, but the bed protection of the navigation channel must be carefully treated due to large current velocities (varying from -6 m/s to +8 m/s). Breakwaters should be built on each side to protect the narrow, long channel. The bed must be protected to increase the bottom roughness and to assistant reducing the surges.

As the high velocities (up to 8m/s) occur in the channel, the bottom protection stability should be further investigated and compared with cost. Additionally, future climate changes may call for a navigable barrier in this channel to be constructed in 20 or 30 years later because of unexpected higher surges. Then a question arises, is this option really technically and economically feasible during its design lifetime? To answer this question, an estimate on the total costs compared with a reference project is performed in this section.

8.3.1 Gate option

If required, a navigable barrier inside the navigational channel could be constructed. Due to shipping strict requirements are imposed on the barrier design. Notes should be paid to the maximum closing frequency and the operation induced translation waves within special limits. The space required for design vessels is prescribed in Section 7.3.1. The minimum width for two-way channel above the local reference sea level is 172m, and the minimum sill depth is WD – 16.25m.

The most popular gates are the swing gates (barge gates, as applied for the Lake Borgne Storm Surge Barrier) and floating sector gates (as applied for the Maeslant barrier). These types have unlimited height clearance, thus are widely used in navigational channels. A barge gate consists of a floating barrier with gates with strong abutment and foundation. The operation is relatively simple. But siltation might be a severe problem for a barrier in the Yangtze River.

The floating sector gate could be another choice. But a major disadvantage of the sector gate is that it cannot sustain high negative hydraulic heads. The river discharge of the Yangtze River is quite large, and then the water level inside the estuary could be higher than that on the sea side. The surges inside the estuary can change rapidly, then a mechanic system is required to make the barrier floated quickly to minimize the negative impacts(Hartsuijker & Welleman, 2007).

Figure 8- 4. Left: Barge gate in the United States; Right: Sector gate in the Netherlands ((J. Janssen & Jorissen, 1992))

8.3.2 Open option

Whether or not to close the navigational section during storm conditions is attractive to costs. The most attractive one is to keep the navigation channel open (no barrier) during all conditions. Water levels and flow velocities inside the estuary are already checked in Section 7.5.2. Result shows, when the width of open navigation is designed as 375m, no flooding occurs inside the estuary. Yet the high velocities (up to 8m/s) occurring in the channel forces the bottom protection stability to be further investigated and compared with cost.

Because of sudden vertically and horizontally restriction of the barrier, erosion of bed materials close to the structures can easily occur. The critical wave load situation is when wave trough occurs at the front of the structure, causing overturning, see Figure 8- 5. The scour protection would be further investigated.

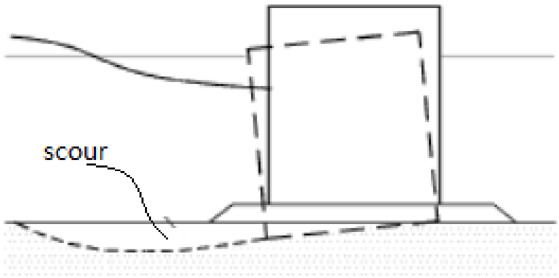


Figure 8- 5. Scouring in front of a hydraulic structure

8.4 Conclusion

In this chapter, the barrier in the South Chanel of the Yangtze Estuary is designed. Two types of functional sections are illustrated.

The environmental section is required to provide opening for water exchange between the sea and river. Thus the main requirements for the environmental sections include preventing flooding during storm conditions and supplying extra opening for water discharge during normal conditions. Due to this reason, lifting gates can be applied, because of the easy construction and rich experience. Then, the main dimensions of lifting gates are calculated. The calculations of top level of closed lifting gates lies in whether or not overtopping is allowed.

The navigational section has also been discussed. Two options are described in general. One is to have an open-closable gate, which is closed during storm conditions, but opened during normal conditions. The gate should first fulfill the navigational requirements as a navigable gate. The other option is to keep the section open under all circumstances. It can reduce costs significantly without gate. However, because of sudden vertically and horizontally restriction of the barrier, erosion of bed materials close to the structures can easily occur. The critical wave load situation is when wave trough occurs in front of the structure, causing overturning.

Three combinations of different sections are proposed, see Table 8- 1. Option 1a consists of 172m-wide open navigational section and 44 lifting gates without overtopping. Different to Option 1a, the opening of the navigational section is extended to 375m, in order to reduce the current velocities through and behind the gap. In Option 2a, a gate is located in the navigational section, while certain level of overtopping is allowed into the estuary. Option 1b and 2b will be discussed further in following chapters, namely 'open option' and 'gate option' separately.

Table 8- 4. Summary alternatives barrier system in the South Channel

Option	Navigational section		Environmental section		Max. water level rise inside estuary	Max. flow velocity through channel	Remarks
1a	Open navigational section	Opening number: 1 Width: 172 m Depth: 16.25m	Lifting gate	Opening number: 44 Width: 80m Height: 19.5m	3.1 m	9m/s	Extra scour protection required
1b	Open navigation channel	Opening number: 1 Width: 375 m Depth: 16.25m	Lifting gate	Opening number: 42 Width: 80m Height: 19.5m	3.5 m	8m/s	Extra scour protection required
2a	Barge/sector gate	Opening number: 1 Width: 172 m Retaining height: 5.1m	Lifting gate	Opening number: 44 Width: 80m Height: 17.6m	3.5 m	-	-

9. DESIGN LEVEL 4.1 – GATED NAVIGATIONAL SECTION

This design level focuses on the design of navigational section of barrier system in the South Channel in the Yangtze Estuary. This chapter discusses the option with movable gate. First, the gate type and materials are determined. Then the structure and stability are checked.

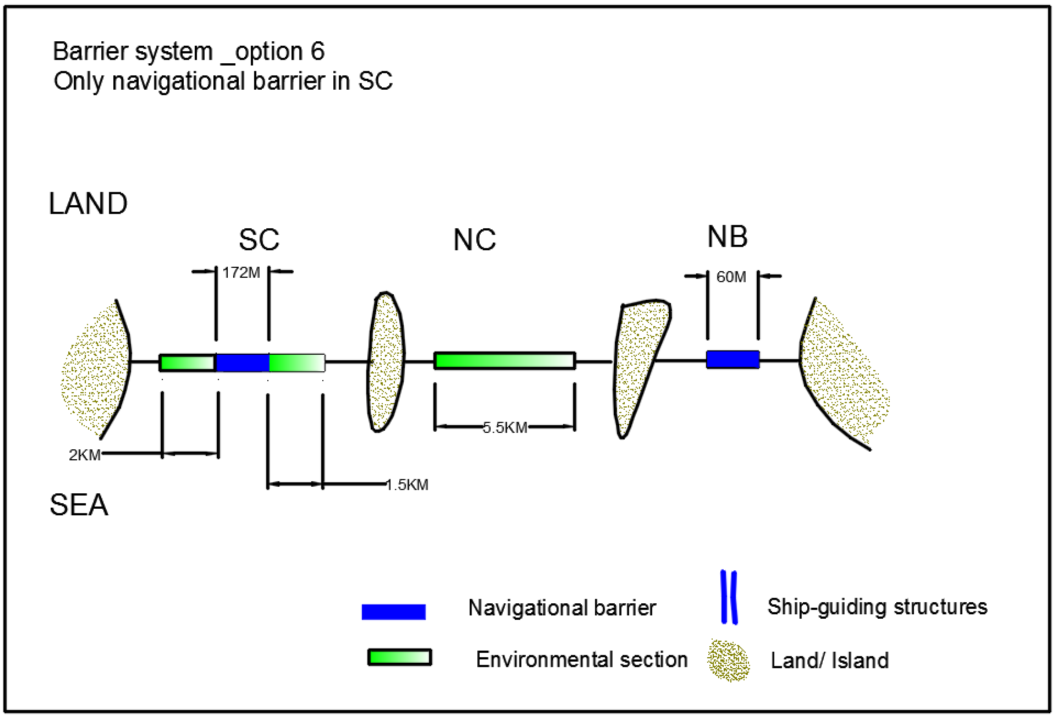


Figure 9- 1. Schematic plan view of gate option

9.1 Gate type selection

This section presents an overview and evaluation of several alternatives for the navigational section. Most of the gate types presented in this part have already been constructed and have a great performance in the past decades. Additional information on these gate types refers to Appendix C.

Similar to the selection of environmental gate type in Section 8.2.1, a first quick evaluation of different gate types focuses on four design criteria. Those criteria are equally weighed. They obviously do not cover all aspects and should not be equally weighed, but it is assumed this set suffices on this level of design. It is recommended to include more criteria and weigh their importance according to different stakeholders' interests.

One of the most important requirement is no delay of shipping is allowed. Thus the selected gate for the navigational section must have unlimited clearance height. Also because the minimum width of navigable is determined in Chapter 7 as 172m, the gate must be suitable to such large span. Four navigable gate types satisfying these requirements will be evaluated in this part:

- Flap gate
- Barge gate
- Floating sector gate
- Inflatable rubber dam

The evaluation of different barrier gates focuses on:

Structural aspects

- Load concentration/ transfer of load to gate supports
- Sensitivity to hydrodynamic and wind load
- Span of the gate

Operation aspects

- Adaptability to negative differential head
- Closure in flowing water and waves
- Effects of silting
- Response for immediate operation when needed

Flexibility and maintenance aspects

- Flexibility for future changes
- Accessibility of inspection and maintenance
- Ease of replacement

Construction aspects

- Abutment and bed protection simplicity
- Construction method and experience with similar projects

To make a more detailed assessment of course it would be the best to take a preliminary draft for all alternatives. A good decision cannot be made just on simply comparing barriers on a few, equally weighed criteria. A detailed multi criteria analysis, along with a cost-benefit are recommended, as is done for flood protection system determination in Chapter 5. However this would be too much time consuming. Thus this barrier gate type selection would keep firefly. The scores on these criteria are specified by YES (+), NO (-), or MEDIUM (±). The result is shown in

Table 9- 1.

Table 9- 1. Scores navigable gate alternatives

GATE TYPES		1	2	3	9
		Flap gate	Barge gate	Floating sector gate	Inflatable rubble dam
CRITERIA	Structura aspects	+	+	±	-
	Operation aspects	-	+	±	±
	Flexibility and maintenance	-	+	+	+
	Construction	-	±	±	±

The flap gate has its disadvantage of being under water, which makes the inspection and maintenance difficult. The possibility of corrosion is high due to that fact, which makes the maintenance cost higher. Silting is also a problem for the flap gate; the accumulation of sand behind the gate may cause problems in operation. Additionally, the flap gate is sensitive to vibrations. The gate cannot function in both directions; it has small stiffness during operation and may be subjective to the waves. However, one advantage is the operation procedure of the flap gate is simpler than other gates. But the flap gate is not widely executed and maybe due to unexpected challenges during the design and realization stage, which will increase the risks of potential errors for this project.

The inflatable rubber dam faces the durability issue because the rubber material. The fabrics may be replaced around every 30 years. Moreover, the rubber layer is quite sensitive to flow induced vibrations, which will possibly happen in this design situation. But these disadvantages can be interpreted as an opportunity to the barrier more flexible to future climate change after 100 years.

In case of the sector gate, the maintenance is not as difficult as the flap gate. The hinged can be located in dry dock making the maintenance easier than underwater. However, the sector gate has the most complicated operation systems. The negative differential head may cause problems for the stability, the possible sediments may cause damage and make the closure procedure difficult and the gate would be sensitive to flow induce vibrations. Regarding to the construction aspect, the sector gate needs too much space for storage in the docks. The sector gate also requires a foundation with high strength because the large force transferred to the subsoil, while in case of Yangtze Estuary the foundation is not too strong. Although the floating sector gates have already been executed already in some places and the knowledge is available, they have the complicated design and construction procedure, which will increase the risk of errors and delays, also the project control is hard in this case.

The barge gate seems to have average higher scores than other gate types. A large span of the barge gates is feasible and gates are suited to reverse differential head and reverse flow during operation. But the negative hydraulic head will cause some problems if the barge gate is not designed well. There is a load concentration and transfer to the hinges, which is rather a disadvantage. In terms of the construction aspect, the barge gate requires less space and the less strong foundation is acceptable. Concerning the construction aspect, the barge gate has moderate construction difficulties. It can be easier to control the project. But they are not executed for the large openings yet, which makes the design and construction challenging, due to this fact this kind of gate scores a "±" on its construction aspects.

The results show the barge gate is the most appropriate gate type for the navigable barrier in the Yangtze River. The floating sector gate is the other option which in on the second place with some small disadvantages. It can be seen with small changes in the analysis, floating sector gate can score better than barge gate. However, the barge gate is about to be further studied in this report.

9.2 Operation concept

In the previous section, the barge gate is chosen as the best option, which may fulfill the design requirements of this project. Because of the poor subsoil condition at selected site, the foundation cost would be decisive. Considering the financial issues of the project, it is beneficial to design the gate as a floating structure, as it reduces the high forces to the weak foundation in this case.

The floating gate is operated in several phases. Next, different operational phases are described, as well as the decisive loading cases. The information is important for structural design in each of those cases.

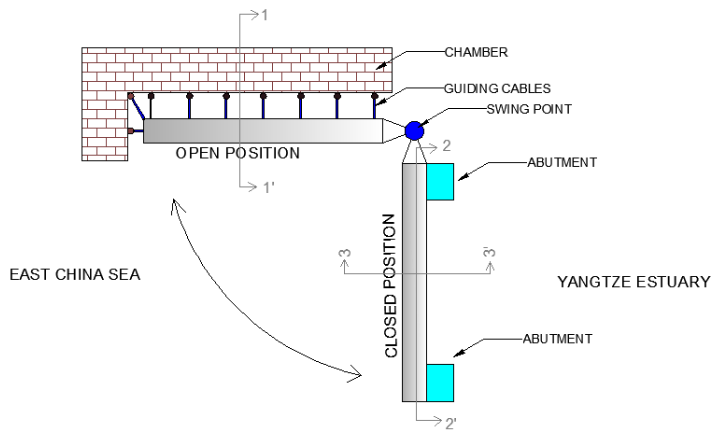


Figure 9- 2. Plan view of the floating barge gate, not to scale (not to scale)

9.2.1 Gate during closure

When a storm is expected, the opening is closed by forcing the barge gate to afloat. The floated gate is rotated around the swing point (articulation system). Initially, the gate is floated without any ballasting. During the closing process, the gate can have more or less drought by adjusting ballasting. The critical situation is when the barge gate is fully empty of ballasting. It is the same in the previous situation.

At the end of closure, the gate will berth horizontally on the abutments. The berthing loads acting against the abutment must be considered in the abutment design.

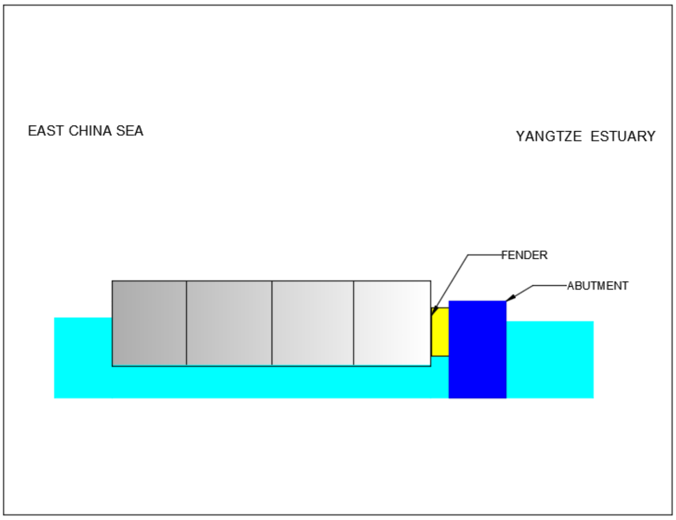


Figure 9- 3. Side view of floating gate during closing process, not to scale

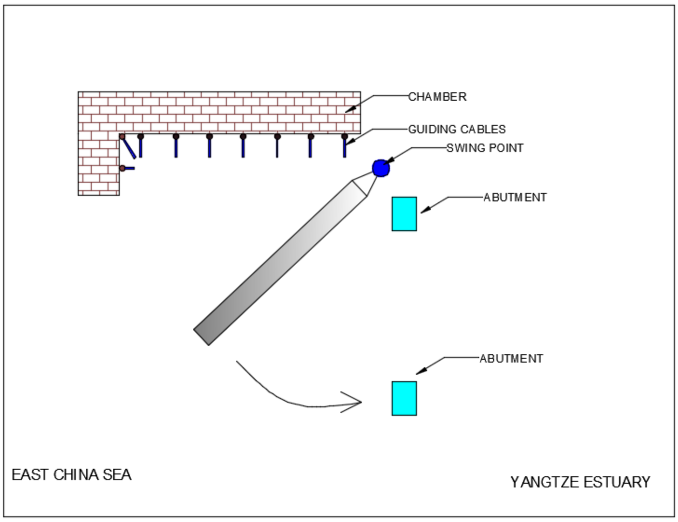


Figure 9- 4. Bird view of floating gate during closing process, not to scale

9.2.1 Gate in open position

In open position, the gate is stored perpendicular to flow direction to allow free shipping under normal conditions. Originally, the gate is ballasted with water and sinks up to certain level to its foundation, see Figure 9- 5. When resting on the foundation, the slab should provide sufficient strength to bear the ballasted water. When a storm is expected, the gate is floated by inflation. The guiding cables help the floating gate stay in its open position in the chamber. The decisive load case in this situation is when it is fully empty. The maximum force acts on the gate due to lack of counter ballasting water weights.

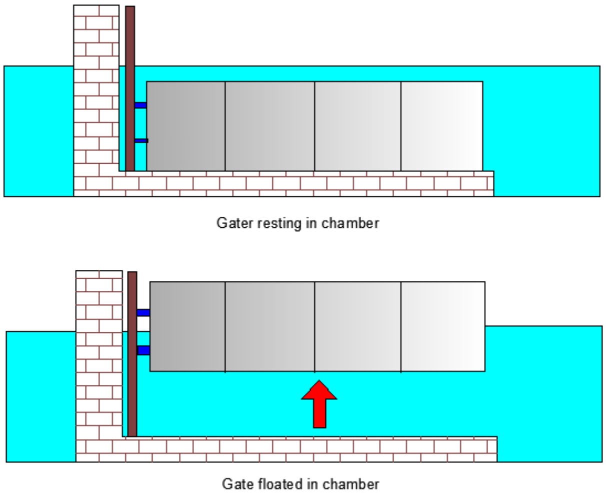


Figure 9- 5. Barge gate in open position, not to scale (cross section 1-1')

9.2.2 Gate during immersion at closed location

After the berthing energy is absorbed by the abutments, the gate rests at its final location. Then the barge gate is ballasted with water and finally sinks to its foundation. This phase is estimated to take around 60 minutes. The immersion process is done with help of compressed air inside the ballast. At the beginning, the gate is allowed to roll on the fender, which is located at the ends of the gate.

The critical situation is when the caisson is ballasted with water, considering different ballasting height. The gate should keep stable during immersion. But the situation might not be as critical as the empty situation, as the ballasting water inside the gate will counter some effects.

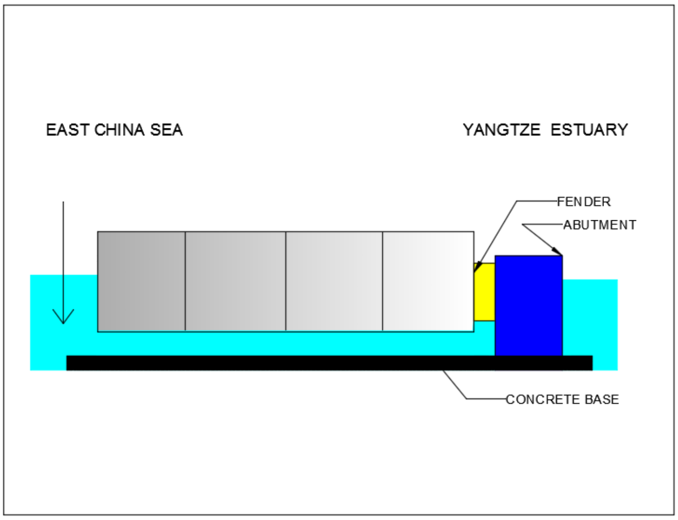


Figure 9- 6. Gate during immersion, not to scale (cross section 3-3')

9.2.3 Immersed gate at closed location

When the gate is totally immersed at closed location, the water levels in the East China Sea and in the Yangtze Estuary can at the same height or different levels. It will be discussed separately as follows.

<u>During normal conditions</u>

When the gate is fully immersed at its final location on the abutments under normal conditions, the water levels inside and outside the barrier are almost the same. The design situation should be the design of the ballasted gate on the foundation vertically only. The gate should be able to resist the vertical loads, including the self-weight of the gate and the ballasting water. In some conditions, to make sure the gate rests on the foundation, the ballasting water is more than required for the design drought. Then the situation becomes more critical.

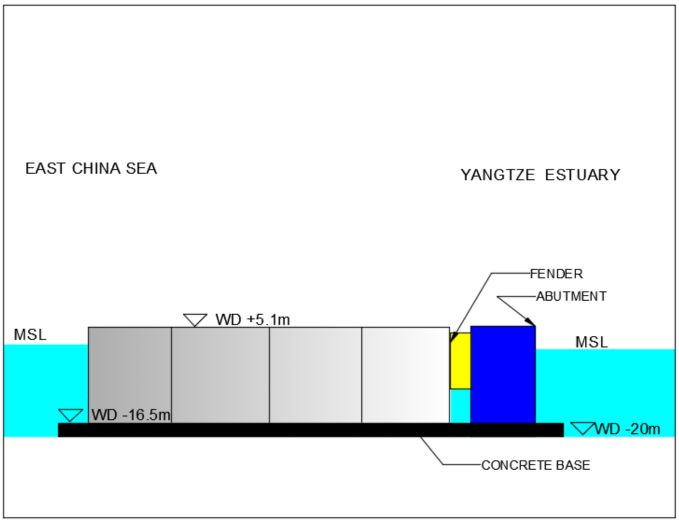


Figure 9- 7. Closed gate during normal conditions, not to scale (cross section 3-3')

<u>During storm conditions</u>

When the ballasted gate rests on the foundation during storm conditions, the design case is more critical than the previous condition. The gate rests vertically on the foundation and horizontally on the abutments. In the overall structural design, the gate will be checked both vertically and horizontally as a beam on two supports.

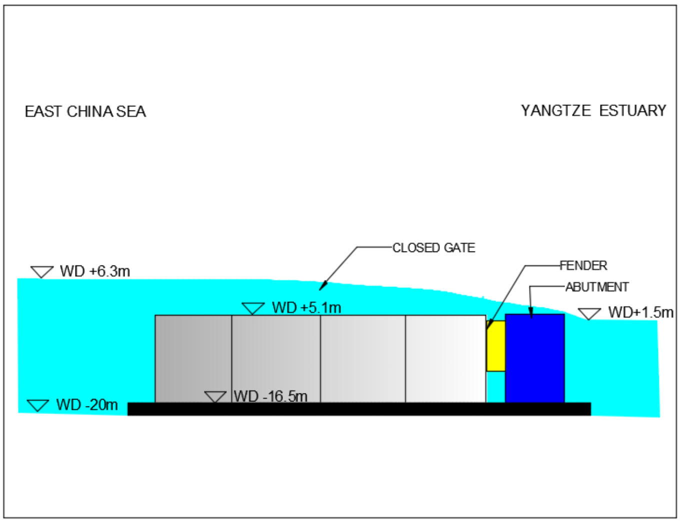


Figure 9- 8. Closed gate during storm conditions, not to scale (cross section 3-3')

9.2.4 After storm condition

After the storm, the gate is floated again and rotated to its original position. It is possible when the water level inside the Yangtze Estuary is higher than the sea level, a negative differential head occurs. If the structure is symmetric, this situation is less critical than during the storm condition, because of a smaller negative head compared to the positive head. After the gate is floated again, the gate will open under the current force.

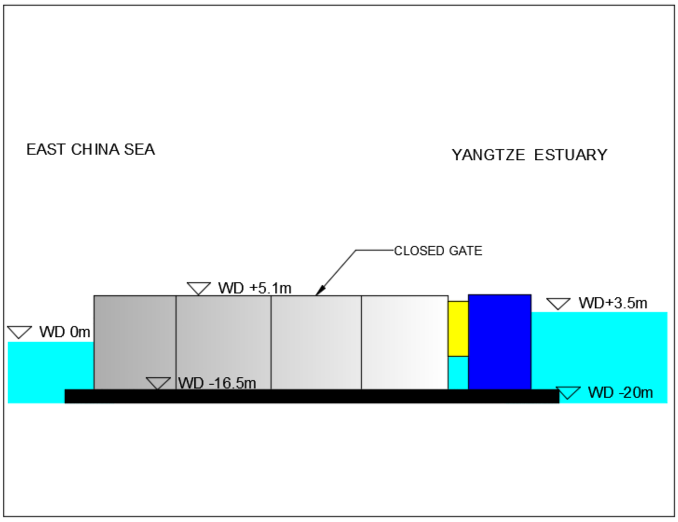


Figure 9- 9. Closed gate after storm condition, not to scale (cross section 3-3')

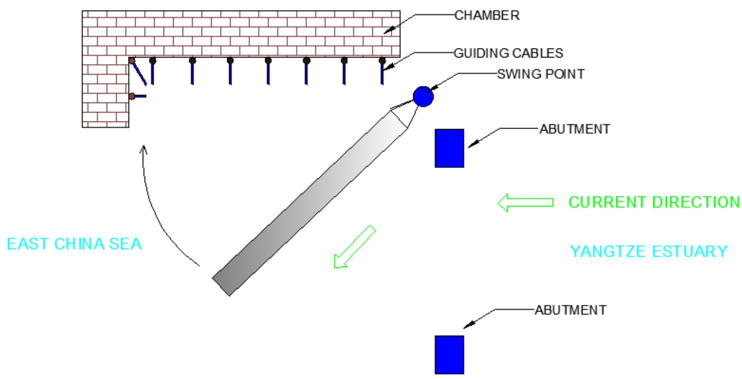


Figure 9- 10. Plan view of floating gate after storm condition, not to scale

9.3 Gate Materials

The barge gate can be constructed with steel or concrete. Several types of construction materials will be discussed in this section.

9.3.1 Material options

Concrete is a common material, which is widely used for the gates of storm surge barriers. This is usually the main structure of the gate, but the moving parts are mostly made from steel.

Steel is the most popular material for barrier gates, due to its high strength to weight ratio. But one disadvantage is that steel-made gate is sensitive to corrosion. Thus, most steel gates will be re-coated during its life-time, contributing high maintenance cost.

High Performance Lightweight Concrete (HPLC) with a high strength to weight ratio, has been studies as a floating barge gate in the Inner Harbor Navigation Canal (IHNC) in New Orleans, United States. HPLC gate is favorable for operability and movability due to its high strength, high workability, high durability but low density. Thus, the HPLC gate can maintain high performance while reduce the self-weight.

Fiber Reinforced Polymer (FRP) is the other possible material, with high strength to weight ratio but low maintenance cost. Kok (2013) has studied how FRP works in large hydraulic structures. The study shows, the larger span is possible(EF Zhang et al., 2012). The limitation does not depend on the material itself but only the manufacturer. Also, the FRP gate is more cost-efficient compared to the steel gate because of low possibility of corrosion.

9.3.2 Material selection

The adverse environment conditions such as chloride corrosion, wave erosion and high sand density are present for the gate. The function and the lifetime will be affected adversely. The floating structure should be easy to move, thus a lighter gate is more favorable.

The concrete can be repaired when the deterioration is started. But the conventional concrete consequently increases the weight and draught of the gate, which will cause problem to the opening and closing process of the barge gate. Indeed, HPLC with long service life and lightweight will be used in this project. Although higher cost compared with normal concrete, but the additional cost can be encountered by the reduction cost due to lower dead weight. But the complicated design requires special attention compared to normal concrete.

One to be noted, in the preliminary design, the gate is designed in HPLC. However, prestressed can also be an alternative. If the gate with HPLC is too heavy, prestressed can be used to reduce the dead weight. The disadvantages are more expensive material and delivery and more complex design procedure. Then the material should be discussed further.

Figure 9- 11. Gulf Intracoastal Waterway Floating Barge Gate made of HPLC

9.4 Gate structure design

In this section, the structural part of the barge gate will be designed. First, the design inputs and assumptions are discussed. Then the gate is designed for different load cases according to different operational phases which are already described in Section 9.2.

9.4.1 Design input

- Hydraulic conditions

Non-storm conditions		
<u>Tidal characteristics</u>	Tidal range (mean) Tidal range (maximum)	2.7m 4.6m
<u>Sea level rise</u> (for 100 year lifetime)	Absolute sea level rising height Land subsidence	0.5m 0.3m
<u>Discharge</u>	River discharge (maximum)	92,600 m^3/s
Storm conditions		
1/1,000 [1/year] return period	Maximum surge level Maximum waver height	WD+ 6.3 m 5.5m

	Significant wave height Peak wave period	3.1m 12s
Positive head	Maximum surge level Corresponding basin level	WD+6.3m WD+2.5m
Negative head	Sea water level Corresponding basin level	WD 0m WD +3.5m

- Material properties

Concrete

The material is chosen as the lightweight concrete HPLC. Lab test shows class C55/67 is available for large scale concrete structures(Li et al., 2000).

The characteristics of concrete are listed as follows:

- Characteristics compressive cube strength f_{ck} = 67 MPa
- Mean value concrete cylinder compressive strength (after 28 days) f_{cm} = 63 MPa
- Mean value of axial tensile strength of concrete f_{ctm} = 4.2 MPa
- characteristics axial tensile strength of concrete (5% fractile) $f_{ctk,0.05}$ = 3.0 MPa
- Secant modulus of elasticity of concrete E_{cm} = 38 GPa

The design values of concrete strength are defined as follows.

Design value of compressive strength of concrete:

$$f_{cd} = \frac{f_{ck}}{\gamma_{c,m}}$$ *Equation 9- 1*

Design value of tensile strength of concrete:

$$f_{ctd} = \frac{f_{ctk,0.05}}{\gamma_{c,m}}$$ *Equation 9- 2*

Within which, $\gamma_{c,m}$ is the safety factor. For compression stress in concrete$\gamma_{c,m} = 1.2$. For tensile force in concrete$\gamma_{c,m} = 1.4$.

Reinforcement

The structure is concrete with reinforcement. For preliminary design, the reinforcement is selected as B500B. The reinforcement can be changed later if required. The main characteristics of steel B500B are:

- Young's modulus E_s= 200,000 N/mm^2
- Characteristics yield strength f_{yk} =500 MPa

9.4.2 Initial gate dimensions

According to previous chapters, the navigable gate thus has a minimum 'wet profile' of 16.25 m depth and 172 m width. The gate top when the barge gate is fully opened is at WD+ 5.1m, which is 0.8m lower the design surge level, because a certain amount of overtopping is allowed in this project for economic consideration. For more information, refer to Appendix K.3.

The gate is actually a caisson, which can be ballasted with water and floated with compressed air. The initial length of the caisson is determined as the minimum required opening plus 4 m on each side to rest on the supports. The initial dimensions of the caisson are determined as listed in Table 9- 2. It is assumed that the caisson is divided into compartments every 12m in the length direction and every 5.5 m in the width direction, which are shown in Table 9- 2 and Figure 9- 12. However, these dimensions might be modified later during the design process to realize optimized design.

Table 9- 2. Initial design parameters of barge gate in reinforcement concrete

Main Dimensions	Length	L_c= 180 m
	Width	W_c= 26.5 m
	Height	H_c= 21.6 m
Thickness Concrete Slabs	External wall	Ww,out= 1.5 m
	Internal wall	Ww,in= 0.5m
	Top slab	Wt= 1.0 m
	Floor slab	Wf= 1.6 m

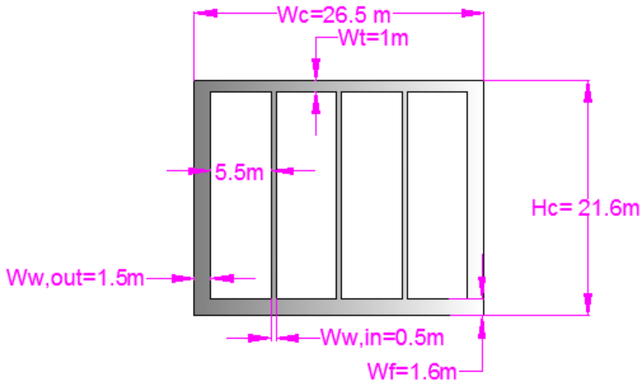


Figure 9- 12. Initial compartment dimensions in the width direction

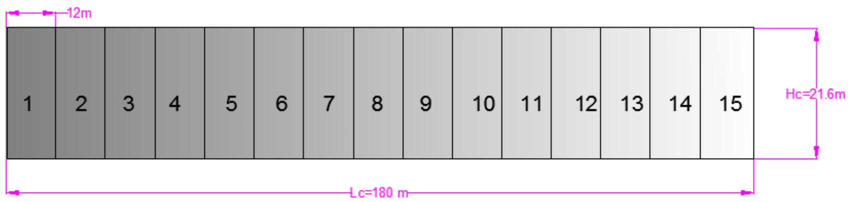


Figure 9- 13. Initial compartment dimensions in the length direction

9.5 Loads and strength description

The potential loads during all operation stages are listed in this part. The strength schemes are also discussed.

9.5.1 Loads schemes

In the preliminary design stage, only the dead weight. Hydrostatic load and wave force are the key working loads on the structure. The dynamic loads will be considered in the stability check stage.

- Dead weight

The dead load of the gate can be simply calculated by the following formula:

$$F_{DW} = \rho \times g \times V \qquad \textit{Equation 9- 3}$$

Within which:

- F_{DW} is the dead weight of the structure [N]
- ρ is the density of the structure [kg/m^3]
- V is the volume of the structure [m^3]

5.7

- Hydrostatic load

The hydrostatic water pressure can be calculated according to:

$$P_S = \rho_S \times g \times h \qquad \textit{Equation 9- 4}$$

The hydrostatic force then can be obtained:

$$F_w = \int P_s dA \qquad \textit{Equation 9- 5}$$

Within which:

- ρ_S is the density of salt water [=10.25 kN/m^3]
- h is the pressure head [m]
- A is the total surface area [m^2]

5.8

- Wave force

The wave can be calculated according to the linear wave theory:

$$P_{wave} = \begin{cases} \rho_w g H_i \frac{cosh(k(d+z))}{cosh(kd)}, & -d < z < 0 \\ \left(1 - \frac{z}{H_i}\right) \rho_w g H_i, & 0 < z < H_i \end{cases} \qquad \textit{Equation 9- 6}$$

Within which,

- H_i is the wave height of incoming wave [m]
- K is the wave number of the incoming wave [m^{-1}]
- d is the water depth [m]
- z is the designed depth [m]

Then the wave force per meter can be obtained as:

$$F_{wave} = \int P_{wave} dz \qquad \textit{Equation 9- 7}$$

- Suction force

If there is head difference at both sides of the floating gate, the so-called suction force can occur due to the differential head and underflow.

The discharge of water under the gate can be calculated according to:

$$q = m\delta\sqrt{2g\Delta H} \qquad \textit{Equation 9- 8}$$

Within which,

- q is the discharge per unit width [$m^3/s/m$]
- m is the contraction coefficient [-]
- ΔH is the differential head [m]
- δ is the height of opening under the gate [m]

The suction force can be obtained(Kolkman & Jongeling, 2007):

$$F_S = C_S \cdot \rho \cdot g \cdot \Delta H \qquad \textit{Equation 9- 9}$$

Where C_S represents the suction coefficient.

9.5.2 Strength requirements

The following checks should be done for different load cases.

- Shear strength check

The shear strength should be checked for slab walls, floors and roofs. The critical shear stress often occurs close to the lower corners. The influence between the bulkheads and the walls are neglected.

The occurring shear stresses read:

$$F_{sh,out} = \frac{1}{2} D_C^2 \rho_{sw} \qquad \textit{Equation 9- 10}$$

$$F_{sh,f} = \frac{1}{2} \rho_C w_t W_C + w_{w,out} H_C \rho_C \qquad \textit{Equation 9- 11}$$

$$F_{sh,t} = \frac{1}{2}\rho_c w_t W_C \qquad \textit{Equation 9- 12}$$

At the initial check, the calculation can be done by comparing the maximum occurring shear force with the maximum allowed shear stress.

The maximum occurring shear stress reads:

$$\tau_{sh} = \frac{3F_{sh}}{2w} \qquad \textit{Equation 9- 13}$$

The maximum allowed shear stress reads:

$$\tau_{max} = 0.4f_{ctm} + 0.15\sigma'_{bmd} \qquad \textit{Equation 9- 14}$$

The criterion reads:

$$\tau_{max} > \tau_{sh} \qquad \textit{Equation 9- 15}$$

Within which:

- F_{sh} is the shear force [N/mm²/m]
- τ_{max} maximum allowed shear stress per meter [N/mm²/m]
- τ_{sh} occurring shear stress per meter [N/mm²]
- w is the thickness of the slab [m]
- σ'_{bmd} is the average design value of concrete compressive strength [N/mm²]

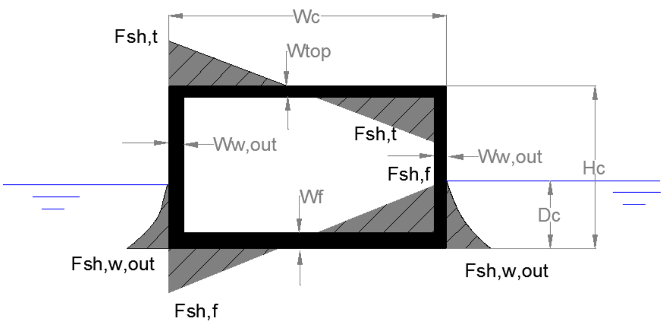


Figure 9- 14. Shear stress scheme on a floating caisson

- Moment capacity

The check on moment capacity can be performed in terms of the required wall and slab thickness. The maximum allowed bending moment can be calculated by:

$$M_U = A_s f_{yd} d\left(1 - \frac{0.52\rho f_{yd}}{f_{cd}}\right) \qquad \textit{Equation 9- 16}$$

Within which:

- M_U is the ultimate bending moment [KM·m]
- A_s is total cross sectional area of reinforcement [m^2]
- f_{yd} is the reinforcement design yields strength [N/mm^2]
- d is the effective height of concrete = h-(c-0.5 ϕ) [m]
- C is the concrete cover [m]
- ϕ is the reinforcement diameter [m]
- ρ is the reinforcement percentage $=\frac{A_s}{A_c}$ [-]

The critical moments appear to occur in the corner of the caisson and in the middle of the floor and top slabs, see Figure 9- 15.

The moments can be obtained by:

$$M_{w,out} = F_{sh,w,out} \times (D_C - 0.5w_f) \qquad \textit{Equation 9- 17}$$

$$M_{f,middle} = \frac{1}{8} \times (D_C\rho_{sw} - \rho_c w_f)W_C^2 \qquad \textit{Equation 9- 18}$$

$$M_{top} = \frac{1}{8} \times \rho_c w_t W_C^2 \qquad \textit{Equation 9- 19}$$

Within which:

- $M_{w,out}$ is the local moment on outer walls [KM·m]
- $M_{f,middle}$ is the local moment in the middle of the floor [KM·m]
- M_{top} is the local moment on top slab [KM·m]

The criterion reads:

$$M_U > M_{ed} \qquad \textit{Equation 9- 20}$$

Where M_{ed} represents the design value of bending moment of the slabs.

For a first estimation, a reinforcement percentage should be chosen. Firstly, the reinforcement if chosen as 1%, then the formula above can be used to determine the required slab thickness, and the reinforcement percentage might be adjusted later.

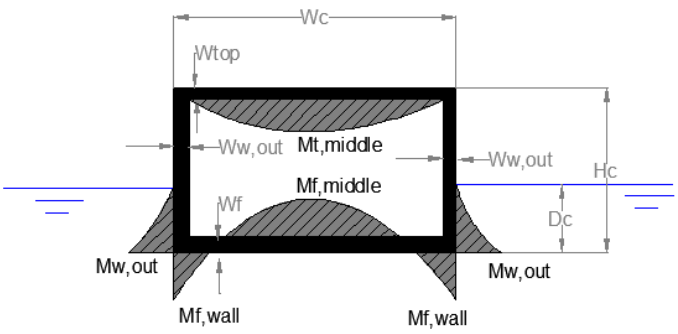


Figure 9- 15. Moment scheme on a floating caisson

- Static stability

To ensure the static stability of a floating element, its minimum metacentric height should be 0.5 m. Before calculation, the metacentric height is defined as follows in Figure 9- 16:

- MG is the metacentric height;
- Point M indicates the point of the intersection of the axis of symmetry, the action line of the buoyant force and the z-axis;
- G is the gravity center including ballasting. It is determined by the position of element's weight. The center point of gravity remains fixed, so it can be regarded as the rotation point as well;
- B is the center of buoyance, where the buoyant force is applied. It equals the gravity point of the displaced water. When the element is titled, the point changes its position.

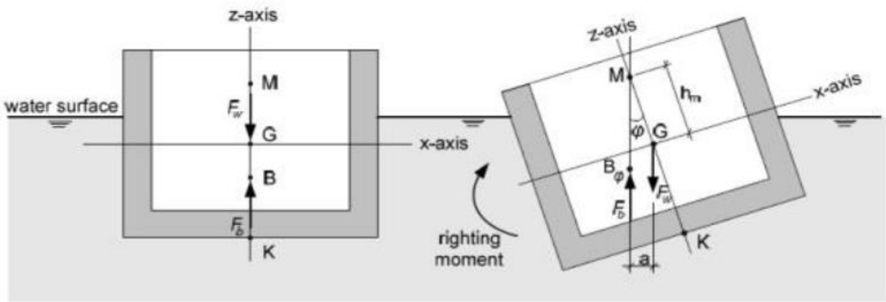


Figure 9- 16. Metacentric height floating caisson (Erfeng Zhang et al., 2011)

Then the mass moment of inertial of the slabs in the x-direction and y-direction, and the volume the displaced water should be defined:

$$I_{xx,floor} = \frac{1}{12} L_c W_c^3 \qquad \textit{Equation 9- 21}$$

$$I_{yy,floor} = \frac{1}{12} W_c L_c^3 \qquad \textit{Equation 9- 22}$$

$$V_{disp} = L_c W_c D_c \qquad \textit{Equation 9- 23}$$

High shear stress occurs due to the floating process beceuse of the hight water pressure utside. Caisson's draft D_c can be calculated by:

$$D_C = \frac{F_B}{L_C W_C \rho_{sw}} \qquad \textit{Equation 9- 24}$$

Within which,

- D_C is the caisson's draft [m]
- F_B is the buyance foce, equals to $F_{v,tot}$ [KN]
- L_C is caisson length [m]

- W_C is the caisson width [m]

Then the dimension can be determined:

$$BK = 0.5D_C \quad \text{Equation 9- 25}$$

$$BM = \frac{min(I_{xx,floor}, I_{yy,floor})}{V_{disp}} \quad \text{Equation 9- 26}$$

$$GK = \frac{0.5H_C V_C - n_x V_{C,in}(0.5H_{C,in} + w_f)}{V_C - n_x V_{C,in}} \quad \text{Equation 9- 27}$$

It is obvious that MG= BK+BM-GK should be higher than 0.5 m.

- Dynamic stability- sway

If the dimensions of the floating structure are too small compared to the length of waves, the structure will sway on the wave. The dynamic stability can be checked by rules of thumb(Liu, Zhu, Wang, Wu, & Wu, 2011):

$$2\pi W_C > gT_{p,reg}^2 \quad \text{Equation 9- 28}$$

And,

$$2\pi L_C > gT_{p,reg}^2 \quad \text{Equation 9- 29}$$

Within which,

- $T_{p,reg}$ is the peak wave period under regular circumstances [s]
- W_C is the caisson width [m]
- L_C is the caisson length [m]

- Dynamic stability- natural oscillation

It can be very dangerous when the period of water movement approaches the period of natural oscillation of the caisson.

Before calculation, the polar inertial radius should be determined first:

$$s_{p-x} = \sqrt{\frac{I_{ZZ,X} + I_{XX,X}}{A_{C,x}}} \quad \text{Equation 9- 30}$$

$$s_{p-y} = \sqrt{\frac{I_{ZZ,y} + I_{XX,y}}{A_{C,Y}}} \quad \text{Equation 9- 31}$$

Within which,

- s_{p-x} is polar moment of inertia radius along x-direction [m]
- s_{p-y} is polar moment of inertia radius along y-direction [m]
- $I_{ZZ,X}$ is mass moment of inertial in zz-direction along x-direction [m^4]
- $I_{XX,X}$ is mass moment of inertial in xx-direction along x-direction [m^4]
- $I_{ZZ,y}$ is mass moment of inertial in zz-direction along y-direction [m^4]
- $I_{XX,y}$ is mass moment of inertial in xx-direction along y-direction [m^4]
- $A_{C,x}$ is area of concrete in x-direction [m^2]
- $A_{C,Y}$ is area of concrete in y-direction [m^2]

Then the period of natural oscillation of the caisson can be calculated:

$$T_{0-x} = \frac{2\pi s_{p-x}}{\sqrt{g h_m}} \quad \textit{Equation 9- 32}$$

$$T_{0-y} = \frac{2\pi s_{p-y}}{\sqrt{g h_m}} \quad \textit{Equation 9- 33}$$

Where h_m represents the metacentric height.

In order to make the caisson stable, the period of natural oscillation should be much larger than that of the waves or swells. The safety factor is assumed to be 2. The criterion then reads:

$$T_0 > 2\, T_{p,reg} \quad \textit{Equation 9- 34}$$

Where $T_{p,reg}$ represents the peak wave period under regular circumstances.

If the resulting natural oscillation period does not fulfill the requirement, one may change the structure. But re-design may not offer an economical solution; the transportation should occur in favorable wave and swell conditions. Then the additional cost can be possibly countered by costs of delays.

9.6 Gate design check

In this section, checks will be done for the structural and stability design of the gate in different operational phases. The calculations are performed in the Serviceability Limit State (SLS) by considering the material factors. For more detailed calculations, refer to Appendix L.1.

9.6.1 Stability check – gate during transport

When the floating gate is during the transportation or during the closing/opening process, the critical case is when the caisson is fully empty. In this situation, water pressure from the outside on the walls and slabs are most critical.

High stress and moments occur due to high water pressure from outside. When the floating caisson is empty, there is no water pressure inside the caisson, so the inner walls are not considered. The effects from the bulkheads are also neglected. This will on the safe side, as the bulkheads will carry some water pressure.

Checks for floating caisson:

- Bending moment check
- Shear stress check
- Static stability check
- Dynamic stability check

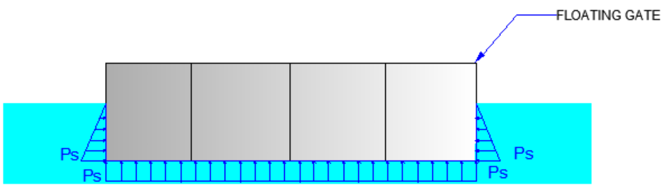


Figure 9- 17. Floating caisson during transport

9.6.2 Stability check – gate during immersion at final location

The caisson is still floating, but some ballast water has been let in to sink the caisson to the desired caisson. Only static stability is checked in this situation. Because shear strength and bending moment for the walls and slabs are obviously more dangerous when the caisson is totally empty, which has already been done in Section 9.6.1.

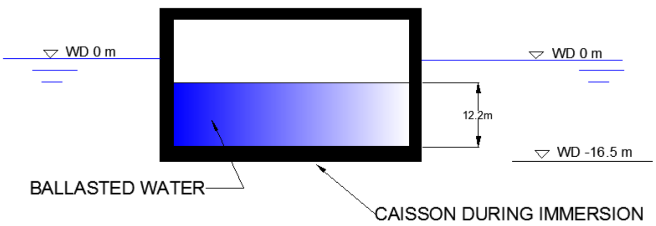


Figure 9- 18. Caisson during immersion to WD-16.5m at final location

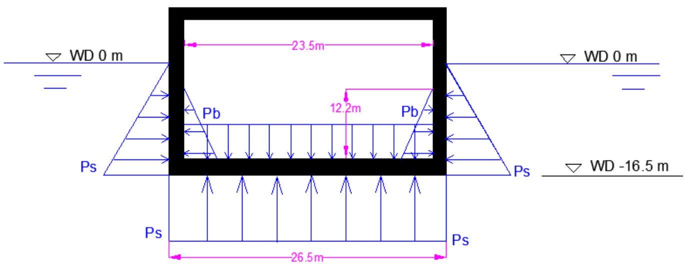


Figure 9- 19. Hydrostatic force scheme during immersion to WD-16.5 m at final location

9.6.3 Stability check – gate just immersed (only ballasted with water)

The design storm condition is the most critical condition when the caisson is ballasted to its final position. The gate will finally rest on the supports, which are located on the ground, but this situation will start as the caisson is just immersed to its design draught. In other word, the caisson can be regarded as a floating structure without supports.

9.6.3.1 Normal conditions

If the gate is immersed during normal conditions, the water levels both in the sea side and the estuary side are almost equal, so the horizontal forces are not considered in this case. The critical situation is when the gate is fully immersed at its vertical supports (foundation). The gate must be able to resist the vertical forces, which are mainly the self-weight of the gate, the weight of ballasted water and buoyancy force. For safety consideration, the ballasted water should be more than required to ensure gate to be stable during storm conditions.

The following checks should be done for this stage:

- Bending moment check
- Shear stress check

The barge gate can be modeled as a beam in this stage, see Figure 9- 20. The uniform load contributing to the beam is considered as the component of the vertical forces. Obviously, the situation is more critical when the caisson is fully empty as discussed in previous part.

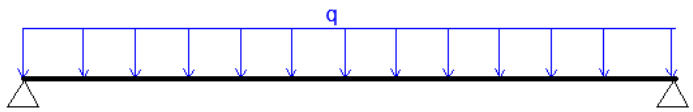


Figure 9- 20. Force scheme of immersed gate during normal condition

9.6.3.2 Storm conditions

In this phase, the loads are shown in Figure 9- 10, including hydrostatic pressure from sea side (P_1), hydrostatic pressure from estuary side (P_2), wave force ($P_{wave,t}$ and $P_{wave,b}$) and hydrostatic pressure to the floor slab. The ballast water inside the caisson also has hydrostatic pressure to the wall (P_b) and to the floor slab (P_b). In this calculation, the wave force is considered as trapezoid distributed.

The following checks should be done for this stage:

- Static stability check
- Bending moment check
- Shear stress check

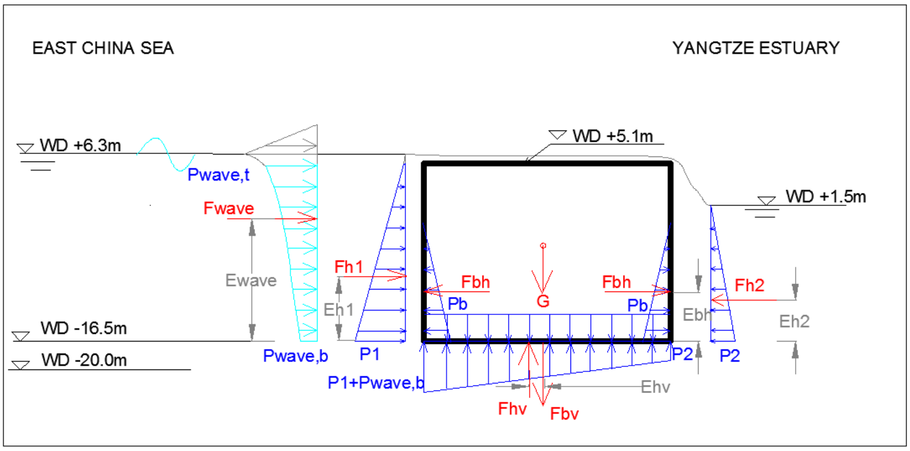


Figure 9- 21. Force scheme of immersed gate due to hydraulic forces (storm condition)

9.6.4 Stability check – gate supported on supports during storm

When the gate is ballasted and finally immersed to its design location, the gate will rest on it supports in both horizontal and vertical direction. In this part, the load cases will be treated separately in both directions.

9.6.4.1 Gate supported on horizontal support

The initial length of the caisson is determined as the minimum required opening plus 4 m on each side to rest on the abutments. When the gate is immersed during a storm, the gate will rest on the abutments in the horizontal direction as shown is Figure 9- 22.

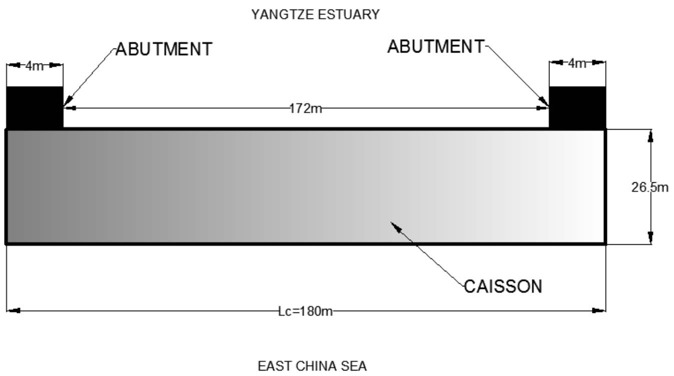


Figure 9- 22. Barge gate rested on the abutments during a storm

The loads in this situation include the hydrostatic pressure from seaside, wave load from sea side (in total P_s in Figure 9- 23) and hydrostatic pressure from estuary side (P_e in Figure 9- 23). The critical load is the highest pressure of the upstream minus hydrostatic pressure from downstream. As can be seen from Figure 9- 24, the largest critical pressure is equal to 120.8 kN/m^2. For a preliminary design, the gate then can be modeled as a beam on supports, with a horizontal uniformly distributed load of 120.8 kN/m^2.

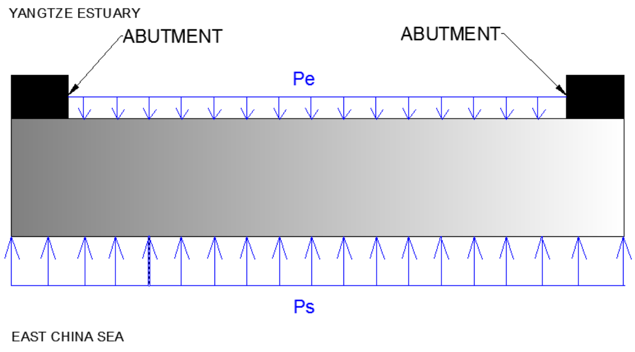


Figure 9- 23. Loads scheme on gate when rested on abutments

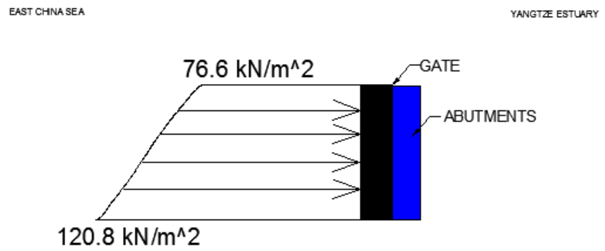


Figure 9- 24. Resultant distributed loads on the immersed gate when gate resting on abutments horizontally

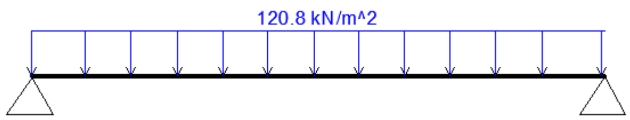


Figure 9- 25. Gate resting on horizontal supports modelled as a beam

9.6.4.2 Gate supported on vertical support

As mentioned in previous part, the initial length of the caisson is determined as 180m. When the gate is resting on the supports vertically, the caisson can be modeled as a beam supported on the foundation, see Figure 9- 26.

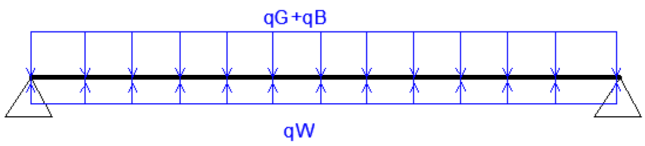


Figure 9- 26. Gate resting on vertical supports modeled as a beam

The loads in this situation consist of the upward force of hydrostatic pressure q_W from downside, downward forces including caisson self-weight q_G and weight of ballasted water q_b.

9.6.5 Stability check – gate at final location after storm

After storm, the gate will be floated again to the initial position. There would be a situation when the water level in the Yangtze Estuary is higher than the sea level. The critical situation is shown in Figure 9- 27. The wave period under regular conditions is considered as 4s coming from the estuary side.

The following checks should be done for this stage:

- Static stability check
- Bending moment check
- Shear stress check

Then the gate is opened under current forces, as the gate will not rest on the supports anymore. Possible damage could be caused due to the potential dynamic vibration. Actions should be taken to reduce negative head force, which will be discussed later.

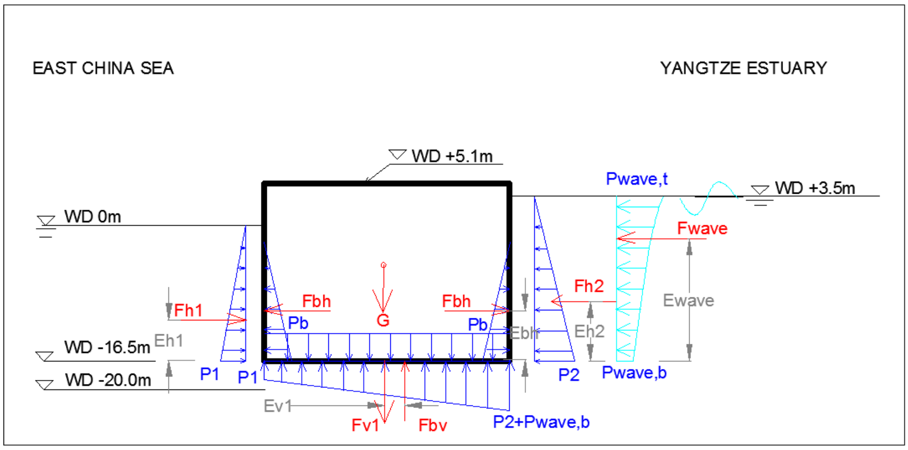


Figure 9- 27. Force scheme of immersed gate due to hydraulic forces (negative head)

9.7 Optimized gate design

According to the calculations in the previous section, the preliminary design of the gate structure can be derived following the same procedure; the final design is shown in

Table 9- 3. Optimizations can be made to in a way that the structure fulfills all boundary requirements, see

Table 9- 4. Two materials are used for the barge gate. Then a simple comparison is performed between these two options.

For more detailed calculations, refer to Appendix L.2.

9.7.1 Optimized gate design in reinforcement concrete

In this section, the results of the calculations and review of stability checks are summarized here. The amount of concrete and reinforcement are designed for the maximum force in this design. More detailed reinforcement map can be calculated with software for further research.

Table 9- 3. Optimized gate dimensions

Main Dimensions	**Parameter**	**Value**	**Unit**
Length Of Gate	Lc	180	m
Width Of Gate	Wc	26.5	m
Height Of Gate	Hc	21.6	m
Thickness Concrete Slabs	**Parameter**	**Value**	**Unit**
External Wall	Wout	1.5	m
Internal Wall	Win	0.4	m
Top Slab	Wt	0.9	m
Floor Slab	Wf	1.5	m
Compartments	**Parameter**	**Value**	**Unit**
Number in length direction	Nx	15	-
Number in Width Direction	Ny	4	-
Draught and Water Height	**Parameter**	**Value**	**Unit**
Initial Draught Of Gate During Transport	Di	6.4	m
Required Ballasted Water Height	Hb	12.9	m
Desiged Balasted Water Height	Hb,S	14.5	m

Stability aspects of the optimized design are checked flowing the same procedure as stated in Section 9.6. The results are summarized in

Table 9- 4 with respect to design requirements:

- Bending moment check. Maximum allowed bending moment should be larger than occurring bending moment of each component, thus, thus$\frac{M_U}{M_{ed}} > 1$.
- Shear stress check. Maximum allowed shear stress should be larger than occurring shear stress of each component, thus $\frac{\tau_{max}}{\tau_{sh}}$ >1.
- Static stability check. The metacentric height of the floating caisson should be larger than 0.5m.
- Dynamic stability check regarding to sway. The dimensions of the floating structure should be larger compared to the length of waves, thus $\frac{2\pi W_C}{gT_{p,reg}^2} > 1$ and $\frac{2\pi L_C}{gT_{p,reg}^2} > 1$.
- Dynamic stability check regarding to oscillation. The period of natural oscillation should be much larger than that of the waves or swells. The safety factor is assumed to be 2. The criteria then reads: $\frac{T_0}{T_{p,reg}} > 2$

Table 9- 4. Stability checks for optimized design

Aspects	Design Parameter	Value	Requirment	
Floating Gate During Transport				
Structural Check	Bending Moment Wall	38.49	Should Be Larger Than 1	Y
	Bending Moment Floor Slab	10.84	Should Be Larger Than 1	Y
	Bending Moment Top Slab	33.53	Should Be Larger Than 1	Y
	Shear Stress Wall	8.07	Should Be Larger Than 1	Y
	Shear Stress Floor Slab	3.16	Should Be Larger Than 1	Y
	Shear Stress Top Slab	4.97	Should Be Larger Than 1	Y
Static Stability Check	Minimum Metacentric Height	1.12	Should Be Larger Than 0.5m	Y
Dynamic Stabiity Check	Sway (Dimensions Of Floating Structure)	2.24	Should Be Large Than 1	Y
	Period Of Natural Oscillation To-X	13.16	Should Be Large Than 2	Y
	Period Of Natural Oscillation To-Y	33.81	Should Be Large Than 2	Y
Gate Immersed At Final Location During Storm (Floating Stucture)				
Structural Check	Bending Moment Wall	1.12	Should Be Larger Than 1	Y
	Bending Moment Floor Slab	1.24	Should Be Larger Than 1	Y
	Bending Moment Top Slab	1.11	Should Be Larger Than 1	Y
	Shear Stress Wall	1.01	Should Be Larger Than 1	Y
	Shear Stress Floor Slab	1.46	Should Be Larger Than 1	Y
	Shear Stress Top Slab	4.48	Should Be Larger Than 1	Y
Static Stability Check	Minimum Metacentric Height	4.84	Should Be Larger Than 0.5m	Y
Gate Immersed At Final Location During Storm (Resting On Abutments)				
Structural Check	Bending Moment External Wall	1.19	Should Be Larger Than 1	Y
	Shear Stress External Wall	1.02	Should Be Larger Than 1	Y
Gate Immersed At Final Location During Storm (Resting On Foundation)				
Structural Check	Bending Moment Floor Slab	1.12	Should Be Larger Than 1	Y
	Shear Stress Floor Slab	1.01	Should Be Larger Than 1	Y
Gate Immersed At Final Location After Storm (Negtive Head)				
Structural Check	Bending Moment Wall	1.20	Should Be Larger Than 1	Y
	Bending Moment Floor Slab	1.02	Should Be Larger Than 1	Y
	Bending Moment Top Slab	1.21	Should Be Larger Than 1	Y
	Shear Stress Wall	1.56	Should Be Larger Than 1	Y
	Shear Stress Floor Slab	1.65	Should Be Larger Than 1	Y
	Shear Stress Top Slab	4.48	Should Be Larger Than 1	Y

9.7.2 Optimized gate design in prestressed concrete

As a design option, the barge gate can also be designed in prestressed concrete. The prestressing type is determined by the construction method. In this case the barge gate is selected to be built with post-tensioning steel with bond. The steel is tensioned after the concrete has hardened. The prestressing force is transferred to the concrete with help of anchorage(PIANC & IMPA, 1997). The anchorage should be covered with concrete to protect from corrosion.

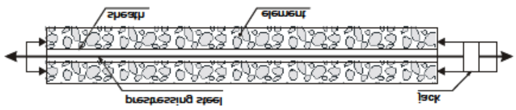


Figure 9- 28. Post-tensioning with bonded steel

Steel Y1860S7 is chosen for design. The total time-dependent loss is assumed to be 20%. Stability checks are done following the requirements in Section 9.6. The calculation is done with equivalent prestressing load method in Figure 9- 29. The external axial prestressing force Pm having an eccentricity e_{p0} at the beam ends.

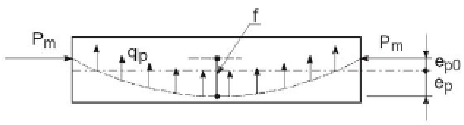


Figure 9- 29. Statically determinate beam prestressed with draped tendons(Lansen & Kluyver, 2006)

The upward uniformly distributed load causing by prestressing force can be calculated:

$$q_p = \frac{P_m}{R} = \frac{8P_m f}{l^2} \qquad \textit{Equation 9- 35}$$

Where,

- R is the curvature of the tendon [m]
- L is the length of beam [m]

The bending moment at the mid-span be calculated:

$$Mp = P_m e_p \qquad \textit{Equation 9- 36}$$

For the prestressed concrete structure, the requirement reads no tensile stress occurs in the concrete. The design criteria read:

Concrete stress at the top fiber in the mid-span of the beam:

$$\sigma_{ct} = -\frac{P}{A_c} + \frac{M_p}{W_{ct}} - \frac{M_G}{W_{ct}} \leq 0 \qquad \textit{Equation 9- 37}$$

Concrete stress at the bottom fiber in the mid-span of the beam:

$$\sigma_{cb} = -\frac{P}{A_c} - \frac{M_p}{W_{cb}} + \frac{M_G}{W_{cb}} \leq 0 \qquad \textit{Equation 9- 38}$$

The variables have been explained in previous calculations.

The results are listed in

Table 9- 1. For detailed calculations, refer to Appendix L.3.

Table 9- 5. Final design of barge gate in prestressed concrete

Main Dimensions	Parameter	Value	Unit
Length Of Gate	Lc	180	m
Width Of Gate	Wc	26.5	m
Height Of Gate	Hc	21.6	m
Thickness Concrete Slabs	**Parameter**	**Value**	**Unit**
External Wall	Wout	1.2	m
Internal Wall	Win	0.3	m
Top Slab	Wt	1.0	m
Floor Slab	Wf	1.5	m
Compartments	**Parameter**	**Value**	**Unit**
Number in Length Direction	Nx	15	-
Number in Width Direction	Ny	4	-
Draught and Water Height	**Parameter**	**Value**	**Unit**
Initial Draught Of Gate During Transport	Di	5.71	m
Required Ballasted Water Height	Hb	13.60	m
Desiged Balasted Water Height	Hb,S	14.00	m

9.7.3 Comparison between two optimized solutions

The barge gate can be designed in either reinforcement concrete or prestressed concrete. The latter has advantages such as more efficient members and lower cracking possibility due its construction method. But the higher costs of materials and fabrication make it probably more expensive.

A rough comparison can be made in

Table 9- 6. The unit costs of each element are adopted from a research report on building materials in Shanghai in the last 10 years(*China Sea Level Change Yearly Repot 2013*, 2013), then exchanged into euros. For example, the price of normal concrete is about 400 Yuan[16] per cubic meter, equals to 50 euro per cubic meter approximately. Because higher complexity while constructing in prestressed concrete, the unit cost of prestressed concrete then is also higher.

The result shows, the dimensions of barge gate are reduced by using prestressed concrete. The weight of gate is reduced from 65,016 tons to 58,230 tons. Obviously, prestressed concrete gate is more expensive as expected. But the less weight and better reliability sufficiently encounter the disadvantage of higher cost. Therefore, as a widely used material in maritime structures, <u>prestressed concrete barge gate is selected for further analysis</u>.

Table 9- 6. Rough comparison between reinforcement and prestressed concrete

		Weight	Unit Cost	Cost			
Normal reinforcement concrete	Reinforcement	107.6 tons	€ 437.5/ ton	€ 47,075			
	Concrete	64908 tons	€ 80/m3	€ 2,284,762			
	Total	**65016 tons**		**€**	**2,331,837**	**¥**	**18,654,696**
		Weight	**Unit cost**	**Cost**			
Prestressed concrete	Steel	53.7 tons	€ 556.3/ ton	€ 29,873			
	Concrete	58176 tons	€ 140/ m3	€ 3,583,642			
	Total	**58230 tons**		**€**	**3,613,515**	**¥**	**28,908,120**

**Note:* *It is important to note that all these calculations are done simply by hand. Further study should be done using advanced software for optimizations, which might lead to probably less thickness of slabs and consequently less weight of concrete. Also, the choice of materials should finally be made by more detailed evaluation including more aspects, such as life-cycle costs, maintenance, and feasibility for future changes. In this study, the full evaluation is too time-consuming, thus the determination is only based on the rough analysis above.*

9.8 Operation system

This movable gate is stored in chamber during normal conditions, and moving to the closed position during storm conditions. Due to the time limit, in this section only a conceptual design will be given for the operation system.

[16] ¥ is the symbol of official Chinese currency YUAN. According to recent exchange rate, 8 Yuan approximates 1 euros.

9.8.1 Guiding system

The barge gate rotates around the swing point during the operational phases. The articulation system should provide free degrees of freedom in all directions while be sufficient to keep the gate stable during movement.

9.8.1.1 Motions of floating gate

In this part, the movements of barge gate during closure process will be discussed, thus the gate can be considered as a floating vessel. A free structure usually has six degree of freedom.

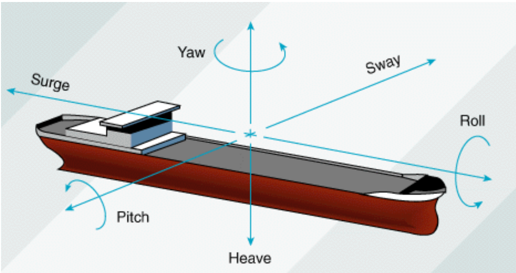


Figure 9- 30. Free floating structure motions modeled as a ship (Bennett Jr & Laborde, 2001)

The possible movements of a floating ship are defined as below:

- **Surge**: horizontal translation along the longitudinal direction of the body. It should be restricted to connect the gate with the swing point.
- **Sway:** horizontal translation along the transverse direction of the body. It should be restricted to prevent moving in the transverse direction during closure.
- **Heave**: vertical translation along the vertical direction of the body. It should be free to provide vertical movement of gate during floating and immersion.
- **Roll**: rotation around the longitudinal direction of the body. It should be free to provide motions of gate during closure and immersion.
- **Pitch**: rotation around the transverse direction of the body. It should be free to provide motions of gate during closure and immersion.
- **Yaw**: rotation around the vertical direction of the body. It should be free to provide rotation around the swing point.

To summarize, only surge and sway should be restricted during gate movements. The designed articulation system should at least satisfy these requirements as analyzed above.

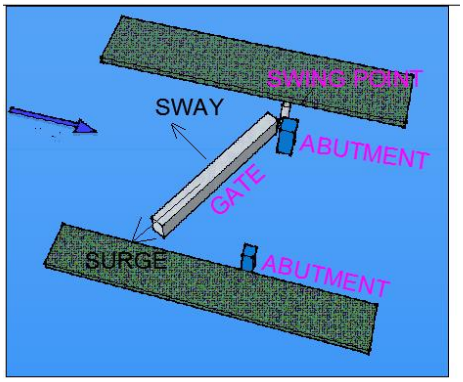


Figure 9- 31. Restricted motions of floating gate during closure

9.8.1.2 Estimations of loads on swing point

According the analysis in previous part, it is assumed motions in all directions are kept free, except the in the sway and surge direction. The displacement in the surge direction can be restricted with help of cables or arms. The articulation system must be able to resist the force in the sway direction, with in turn will help the gate to close.

In this project, the barge gate is loaded by environmental forces induced by waves, currents and wind. In generally, the wave force is more important for floating structure movements than other aspects. Also it can be expected that, the long wave has not strong effects on the gate motions, because the closure runs fast, and the gate has enough time to adjust to the movements of water plane. So the shorter wind waves are much more important for this initial calculation. The forces will be calculated roughly to give an indication of the resulting force acting on the articulation system.

The steady environmental forces can be calculated as below:

Wave force:

$$F_{wa} = \frac{\rho_w g L}{16}\left((1 - c_t) H_{sh}\right)^2 \qquad \textit{Equation 9- 39}$$

Current force:

$$F_c = \frac{\rho_w c_s A_C U_C^2}{2} \qquad \textit{Equation 9- 40}$$

Wind force:

$$F_{wi} = \frac{\rho_{air} c_s A_w U_w^2}{2} \qquad \textit{Equation 9- 41}$$

The variables in the equations are:

- c_t wave height transmission coefficient [-]
- H_{sh} significant wave height [m]
- L length of structure in the force direction [m]

- c_s shape coefficient [-]
- A_C structure area in the direction of current force [m^2]
- A_w structure area in the direction of wind force [m^2]
- U_C current velocity [m/s]
- U_w wind velocity [m/s]
- ρ_w water density [kg/m^3]
- ρ_{air} air density [kg/m^3]

Except the steady forces, the dynamic forces induced by the oscillatory motions should also be considered. A hydrodynamic analysis is required to calculate the motions of gate. However, at this initial design stage, the motions related properties of the gate are unavailable. Being half of the total steady force seems a reasonable assumption for a rough estimation.

For detailed calculation, see Appendix M. The results are as follows:

<u>Total force:</u>

- Sway direction 4090.7 kN
- Surge direction 555.0 kN

It is obvious that the force in the sway is much larger, because of much more resistance in the sway direction. These rough calculations should be modified by more advanced hydrodynamic analysis. The exact gate motions and structural resistance should be taken into account for the design of articulation system. This part is not treated in this thesis.

9.8.1.3 Proposed solutions

Design of the articulation system requires fully acknowledge of the caisson's motions, and the parameters of the overall forces on the swing point during all operational stages. Here, it is difficult to perform the hydrological analysis in this thesis, so the solution will be decided according to as-built projects.

One can refer to the Maeslant barrier in the Netherlands. The articulation system is a ball and socket system. The barrier consists of two floating sector gate, each with a length of 208m and a gate height of 22m. The gate arm is connected to the ball joints. Also, a floating breakwater built in Monaco has a similar system. The articulation system is designed to resist loads up to 100,000 kN. The load cases are similar to this project, so it is possible to construct such a ball-joint system corresponding to this large floating barge gate.

Figure 9- 32. Ball-joint system under construction of the Maeslant barrier(J. P. F. M. Janssen)

Another solution can refer to the articulation system applied in the Bayou Lafourche swing gate. It has a swing arm pivot system, consisting of the pivot assembly and the support arm. This system allows three degrees of freedom just as the system for the current project. But the Bayou Lafourche swing gate is only 25 meters long, which is much smaller, compared to this project. So weather this kind of system can be applied to such large floating barge gate still requires more research.

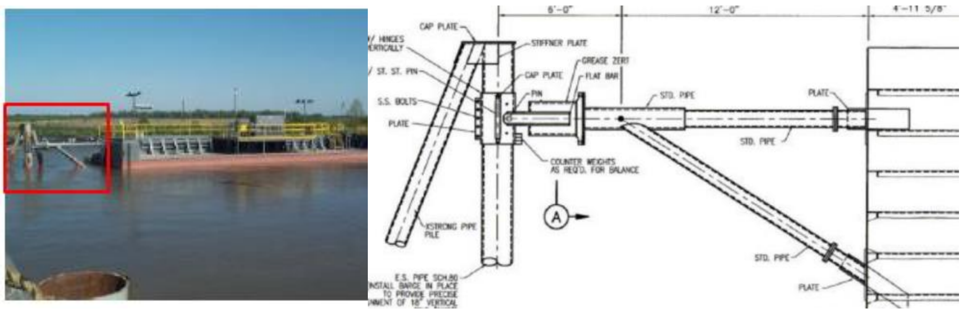


Figure 9- 33. Articulation system of the Bayou Lafourche swing gate (Labeur, 2007)

Except the mechanical parts around the swing point, the spring cables and guide columns in the chamber can be applied to make the gate more stable during movement. The spring cables can keep the gate in close contact with the columns acting like a guide.

However, the design of articulation system requires more accurate hydrological analysis and mechanical checks. Also, how to protect the steel-made structures from corrosion should be noted. These are not the main scope in this thesis, so it will not be further treated.

9.8.2 Gate recess

During most of time, the gate is stored in the gate chamber. The design of chamber should consider the gate material, construction feasibility and future maintenance.

Once the storm is gone, the gate will be floated and moved to its open position again. The chamber design is important for the whole project, as the gate will spend most of its lifetime there. For maintenance purpose, a dry chamber should be built. A small UHMPE lifting gate will act as a door for the barrier large recess, see Figure 9- 34. This will allow for the chamber to be sealed and pumped dry when the gate is not in operation. One key requirement for the chamber is that, the base slab should be able to sustain such heavy gate. Then the vertical force will be transfer to the foundation, which will also be expensive. Another factor influencing the structure design is that, an approaching channel for vessels should be constructed in front of gate, which will also increase the total cost.

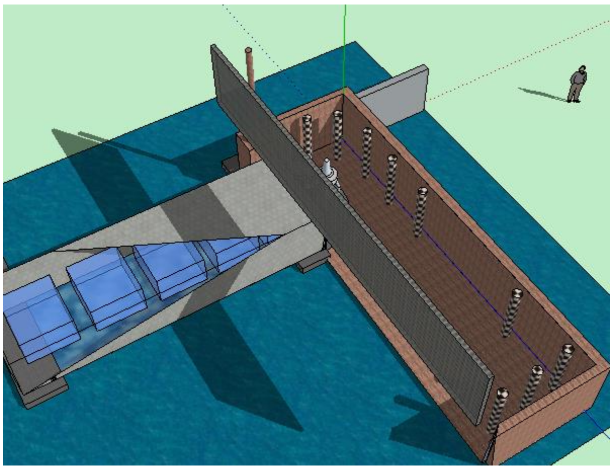


Figure 9- 34. Designed dry chamber for barge gate when not in operation

Considering the disadvantages mentioned above, another option is proposed. When taking a look at the already-built large barrier system, like Easter Scheldt barrier is the Netherlands, some artificial islands were constructed as a connection of the whole barrier system (Figure 9- 15). The main advantage for these islands is that, they provide construction pit for heavy elements, and also space for gate recess. If the island is appropriately designed, it can also work as a bed protection measure. On the other hand, it must be invisible for captains to escape ship corruption. However, despite of such benefits it creates, the disadvantage may make it unworkable for the Yangtze Estuary. The local conditions require as less civil works in water as possible. For the construction of an artificial island, extra docks and more restriction of waterway are required. Thus, whether to construct a pre-fabricated concrete chamber or an artificial island depends on lots of factors, then the final decision requires further estimation. For this thesis, it is assumed that the artificial islands will be built, which will serve as the construction pits for heavy elements, like the concrete slabs, large bed protection elements. But the detailed design is not included in this thesis.

Figure 9- 35. Left: artificial Island in the middle of Eastern Scheldt barrier; Right: chamber of Maeslant barrier built on land

9.9 Abutment design

Abutment as a supporting structure is an important part of the swing barrier. A preliminary design of the abutments is treated in this section. The requirements of the swing barrier from previous chapters are set as the same. As a starting point, the main loads on the abutments will be demonstrated and then the structural design will be performed. Refer to Appendix M for further calculations.

9.9.1 Initial abutment design

Large amount of loads on the gate are transferred to the abutments. To make it stable, a foundation should be sufficient to resist the loads. Two solutions are proposed. One is designed as a pier on a floor slab with light reinforcement concrete to reduce the dead weight. The other solution is to take advantage of prestressed concrete.

As a starting point, the dimensions are estimated to be:

- Height of abutment H_b= 23m
- Width of abutment W_b= 6m
- Length of abutment L_b= 8m

The following calculations will be based on this assumption.

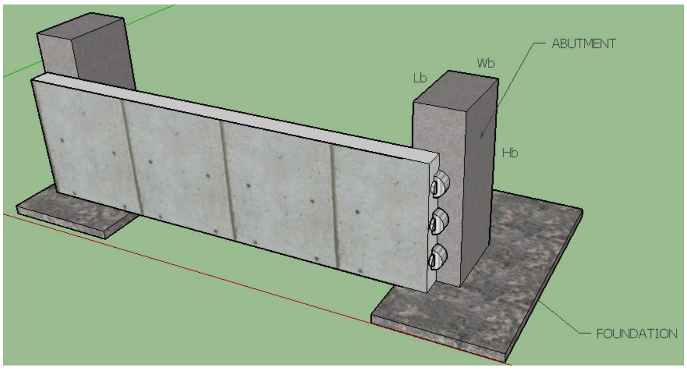


Figure 9- 36. Overview of gate resting on abutments (not to scale)

9.9.2 Structural checks and optimized design

In this part, the major forces on the abutments will be discussed. Then two solutions of normal reinforcement concrete and prestressed concrete will be given.

- Hydraulic force during storm

Hydraulic force, including hydrostatic pressure and wave force in different conditions is the major load acting on the gate. Then the force is transferred to the abutments. As a preliminary design, the abutments are regarded as the supports of the "gate beam", see Section 9.6.4.1.

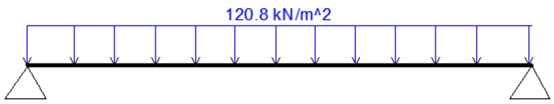


Figure 9- 37. Maximum hydraulic force on the gate during design storm

- Berthing force during gate closure

During closure, the swing gate will berth on the abutments with heavy loads. Berthing energy will be absorbed by fender located at the end of gate. To calculate the berthing force, firstly the fender type should be selected. The berthing system would be a complicated system. In this project, the widely used wheel fender is selected for following calculations. One advantage of the wheel fender is that the relatively soft material can help absorb berthing energy. Also the rolling wheel can reduce friction when the gate is immersing or floating along the abutments. 250-100WF of wheel fender is applied. The high capacity of energy absorption and low maintenance cost make this type suitable.

The gate berthing on the abutments can be regarded as the ships berthing on the lock. Thus, the force on a liner elastic structure can be calculated with(Erfeng Zhang et al., 2011):

$$F = \sqrt{2kE_{kin}} \qquad \textit{Equation 9- 42}$$

Where

- E_{kin} is the total amount of kinetic energy to be absorbed by fender [kNm]
- F is the berthing force from the gate [kN]
- k is the structure stiffness [kN/m]

E_{kin} can be calculated:

$$E_{kin} = m_g V_g^2 C_H C_S C_E C_C \qquad \textit{Equation 9- 43}$$

Where

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

These coefficients will be discussed hereby.

Structure stiffness k. The stiffness of the structure depends on the structure and fenders. Assuming the fenders are normative and the abutments (including foundation) are very stiff.

Gate velocity V_g. The berthing speeds depend on the gate dimension, the load type and the berthing conditions. Knowing the gate weight, the velocity can be approximated from Figure 9- 38.

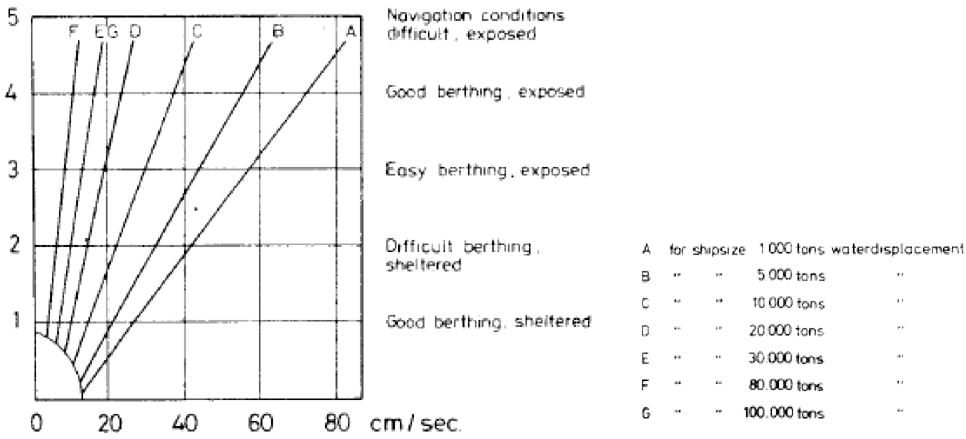


Figure 9- 38. Berthing gate velocity

Hydraulic coefficient C_H. It can be calculated by

$$C_H = \frac{m_g + m_w}{2m_g} \qquad \textit{Equation 9- 44}$$

While the additional water mass can be calculated by

$$m_w = \frac{\rho L \pi D^2}{4} \qquad \textit{Equation 9- 45}$$

Where

- L is the gate length [m]
- ρ is the density of sea water [kg/m^3]
- D is the initial draught of the floating gate [m]

Eccentricity coefficient C_E. This coefficient takes into account the energy dissipation. It can be calculated by

$$C_E = \frac{k^2 + r^2 cos^2(\gamma)}{k^2 + r^2} \qquad \textit{Equation 9- 46}$$

Where

- K is the gyration radius (K= (0.19C_B+0.11) L , C_B= I/(LBD))
- r is the radius between the mass center and the point of collision
- γ is the angle between r and gate velocity

Softness coefficient C_S. This coefficient depends on the stiffness of the abutments and take into account part of the gate's shell. In this project, this coefficient is assumed to be 0.9.

Configuration coefficient C_C. This coefficient considers the hydrodynamic friction induced by the mass between the abutments and the gate. For a preliminary design, this coefficient is assumed to be 1.0 for design purpose.

The detailed calculations can be found in Appendix M. Results show that the berthing force on each fender is equal 486.7 kN, assuming two fenders in the vertical direction in the height of 7m (~initial draught of gate 6.8m) of the abutments.

After the calculations, two designs have been realized. The results are summarized in Table 9- 7. Obviously, due to the lighter weight and better protection from corrosion, prestressed concrete is more favorable.

Table 9- 7. Results of abutment dimensions

		Normal	Prestressed
Abutment height	Hb	23 m	23 m
Abutment length	Lb	8 m	6 m
Abutment width	Wb	6 m	5 m
Total weight per abutment	Wtot	1932.6 ton	1206.0 ton

9.10 Foundation overview

As a supporting structure of the gate and abutments, foundation is another important element in this design. However, the soil conditions of the selected barrier location are not completed. Further survey is required for more detailed information.

The dense layer of sand of the selected foundation location is from almost WD -60m. That makes the design limited to following options:

- Shallow foundation using complete soil replacement
- Preloading the subsoil
- Deep foundation with piles

The first solution seem to be simple, but actually to be quite expensive. It concerns all of the costs for dredging, dumping and refill the pit with sand or gravel as well as transport fees. One to be noted is that, soil settlement still remains although it is not as high as in clay layers. The dumped sand or gravel is not compacted yet. The settlement cannot be neglected.

The vacuum preloading helps speed up consolidation but not yet been applied widely. With the deep-driven piles, the weak subsoil layers are bypass. Ideally all of the piles will be placed directly underneath the slab base, and then the loads are sufficiently transferred to the bearing subsoil. Thus a deep foundation is probably the most cost-efficient solution in this project. The design of deep foundation is not considered in this thesis because of the time limits.

However, one to be noted is that the foundation design largely depends on the barrier type. In reality, the gate type should be reassessed with corresponding foundation method. This report will proceed with a deep foundation method.

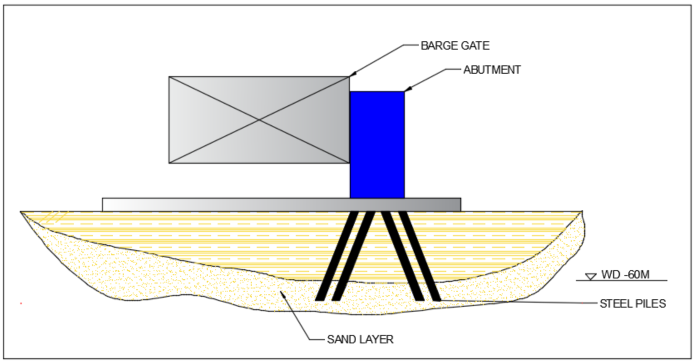


Figure 9- 39. Foundation overview

9.11 Bottom protection

The barrier is built at the mouth of the Yangtze Estuary, which is both tides and river dominated. During normal conditions or a river flood is expected, all the environmental and navigational sections are opened. The large river discharge coming from the Yangtze River will wash out sands, due to the 40% constriction of the original outflow area. A bottom protection is placed which withstand the current and has to protect the riverbed from erosion.

The critical situation is when the normal tides (tidal range 2.7 m) coming from the sea, along with 0.1% river discharge Q_r=92,600 m^3/s coming from the Yangtze River, flow through the barrier together. The discharge through openings and thus current velocity can be performed by using 'storage basin' model, which is the same as done in Section 7.3. According to model, the velocity in the seaside of the barrier will reach 1.3 m/s during normal conditions, but rise up 4.2m/s when floodwater comes down from the river. <u>Then bottom protection should be designed for both sides over the whole barrier.</u>

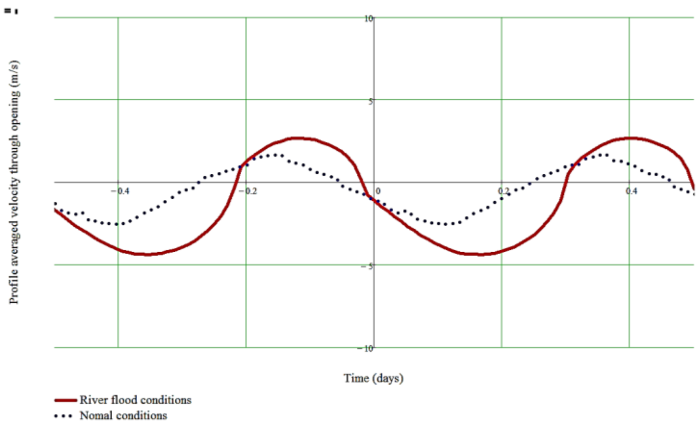


Figure 9- 40.Profile velocity through opening when all gates are opened (positive direction: from sea to the estuary)

For the first estimation, the protection length is assumed to be 10 times the water depth, which is around 200 m. The protection area is divided into two parts (B1: 0-120m and B2: 120-200m) according to the depth-averaged velocities distribution. According to Pilarczyk equation[17], the required rock sizes of top layers are Dn50=0.93m for area B1, Dn50=0.53m for area B2. Then several granular filters can be designed following the filter rules[18].

Next step is to find whether the protection length is enough of not. Two types of calculation on scouring depth are performed for two different design conditions. During normal condition, the current velocity of 1.3 m/s will last almost for the whole lifetime of barrier, so the equilibrium scouring depth is calculated as $H_{se,e}$= 18.5 m. When the floods come from the Yangtze River, the scour development with time should be check within 3 days, which is a reasonable influence period for Yangtze River floods. The maximum scour depth is $H_{se}(t)$=17.9m. Thus, the first design situation dominates. The maximum scour depth behind the 200 m long protection is 18.5m. Taking equation 10-6, the upstream slope of scour hole is 1:2. It is assumed that the sand is loosely packed. That means, the danger of flow slides exists, with very gentle slope (order of magnitude 1:15). With a slope of the scour hole itself of 1:2, the flow slides would damage the structures (18.5*15 > 200+18.5*2). Then the bottom protection should be modified.

Repeating the same design process, a bottom protection of 300 m length is enough to keep the scour in a safe distance to the structure. More information about top layer and filters are presented in Figure 9- 41.

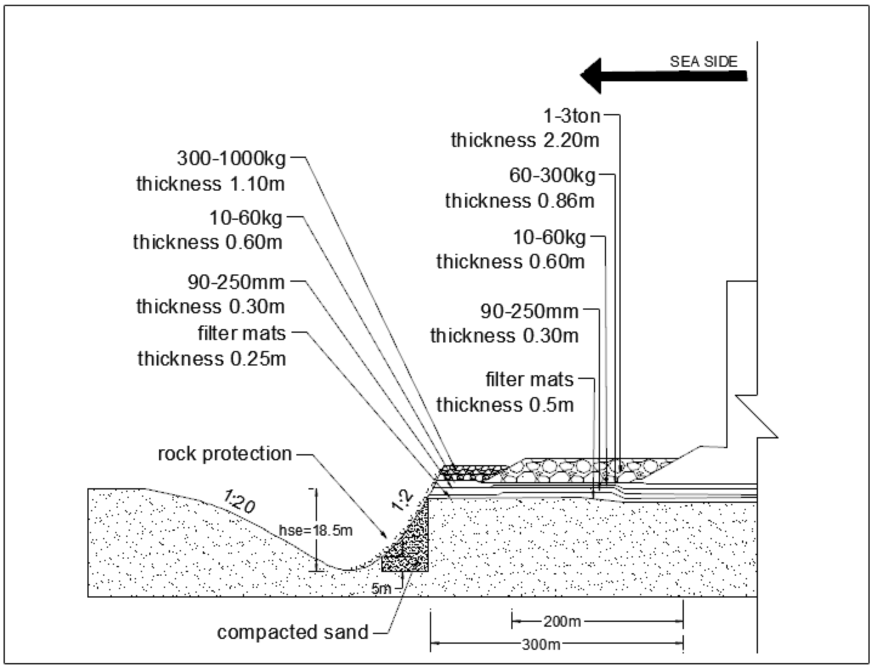


Figure 9- 41. Section view of designed bottom protection at both sides over the whole barrier length

[17]Equation 10-2, see Section 10.3.2
[18] Filter rules for geometrically closed filters (Schiereck, 2001)

9.12 Seepage cut off

A groundwater flow under or along the structure can develop when a head over a structure is present and the seepage distance is too small. Two empirical formulas are used to calculate he minimum seepage distances, which are the formulas of Bligh and Lane. The formulas of Bligh and Lane are presented in Equation 9-51 and 9-52.

$$\sum L_H + \sum L_V \geq C_B \Delta H \qquad \text{Equation 9- 47}$$

$$\frac{1}{3}\sum L_H + \sum L_V \geq C_L \Delta H \qquad \text{Equation 9- 48}$$

Where,

- $\sum L_H$ is the horizontal seeping distance [m]
- $\sum L_V$ is the vertical seeping distance [m]
- C_B is the Bligh coefficient =18 (Very fine sand/ silt/ sludge)
- C_L is the Lane coefficient =8.5 (Very fine sand/ silt/ sludge)
- ΔH is the water level difference [m]

The formulas are used to determine the length of the sheet pilling which is necessary to prevent erosion underneath and along the barrier.The water level difference is determined using the design water level at the downstream side of the barrier, which is equal to the design water level at the sea, and the normal water level at the upstream side of the barrier. The design water head is 6.3-1.5=4.8m. The minimum seepage length for Bligh and Lane are presented in Table 9- 8 for seepage under the barrier. The seepage screen should have minimum length of 77m in the vertical direction according to the seepage formula of Bligh. The proposed seepage cut-off screen is shown in Figure 9- 42.

Table 9- 8. Seepage length for piping under barrier

Aspect	LH	LV	Total seepage length	Hydraulic coefficient
Seepage length (Bligh)	48m	77m	125m	1/26
Seepage length (Lane)	16m	70m	86m	1/17

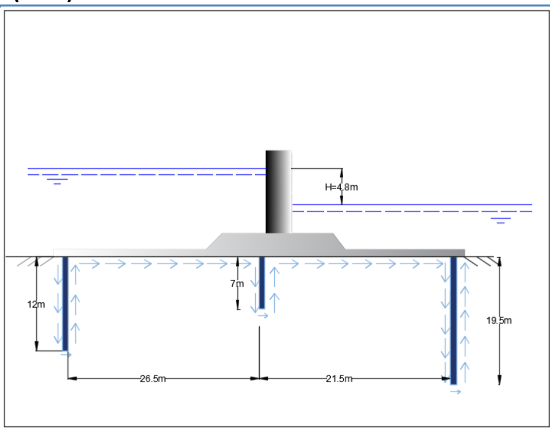


Figure 9- 42.Cross section of barrier and seepage screens

10. DESIGN LEVEL 4.2 –OPEN NAVIGATIONAL SECTION

This chapter represents as another option for the navigational section of barrier system in the South Channel in the Yangtze Estuary. Chapter 9 proposed a normal solution with movable gate, which can possibly be a barge gate as described before. It has been discussed that an 'open option' without gate might also be feasible. But the very high current velocity though opening during floods could destruct the whole structure. The required bed protection for the open-option will be discussed here.

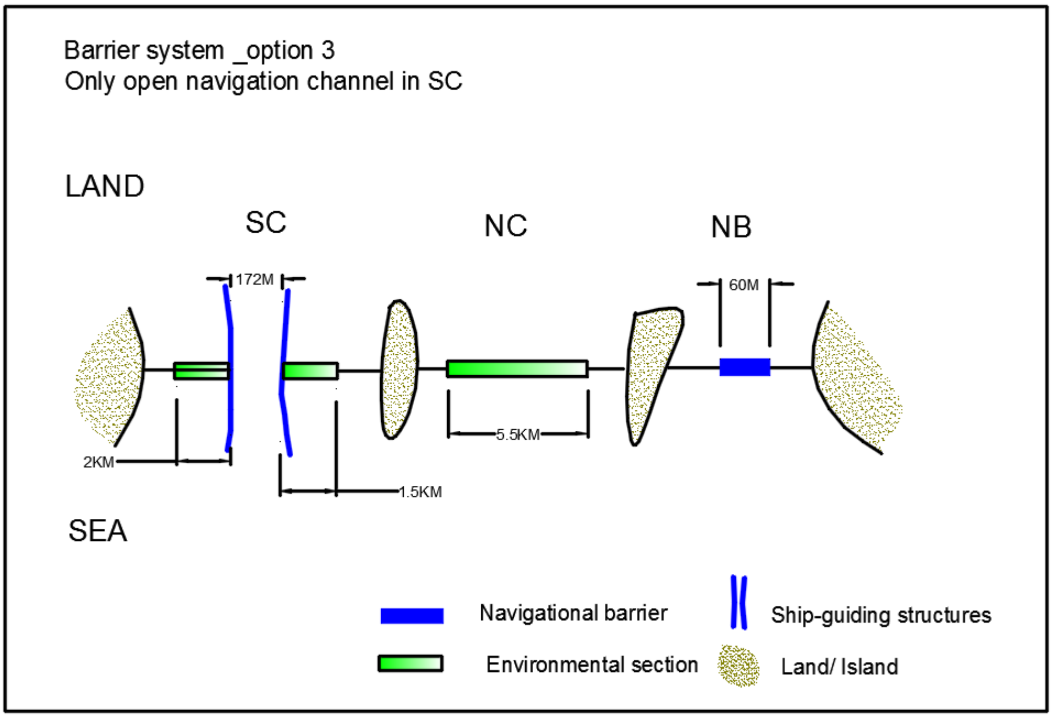


Figure 10- 1. Schematic plan view of open option

10.1 Introduction

The proposed barrier in the South Channel is composed of a navigation channel and environmental barriers, which is already illustrated is Section 7.5. The total span is 5km at this location, with 375 m navigation channel and a 5km environmental barrier, as shown is Figure 10- 2.

Normal solution is to design a movable gate in such systems. According to experience, the gates are always the main part of the total construction cost. It could be interesting to discuss whether the gate is a must or not. Then an open-option is proposed for this project. In this open-option, the 375m wide channel is kept open during all circumstances. As a primary flood defense, two main criteria must be followed for this option:

- **Flood protection**
  The maximum water level inside the Yangtze Estuary must not exceed WD +3.5m. The calculations in Section 7.5.2 show, this safety requirement is met when a design storm is expected.
- **Structure stability**
  When the flood arrives, the huge amount of floodwater can only pass through the relatively small opening. High current velocities occur up to 7.8 m/s. Due to the sudden constriction of waterway, high turbulence shows up as well. Then scouring can be a vital damage to the stability of the whole structure. The bed protection should be carefully designed.

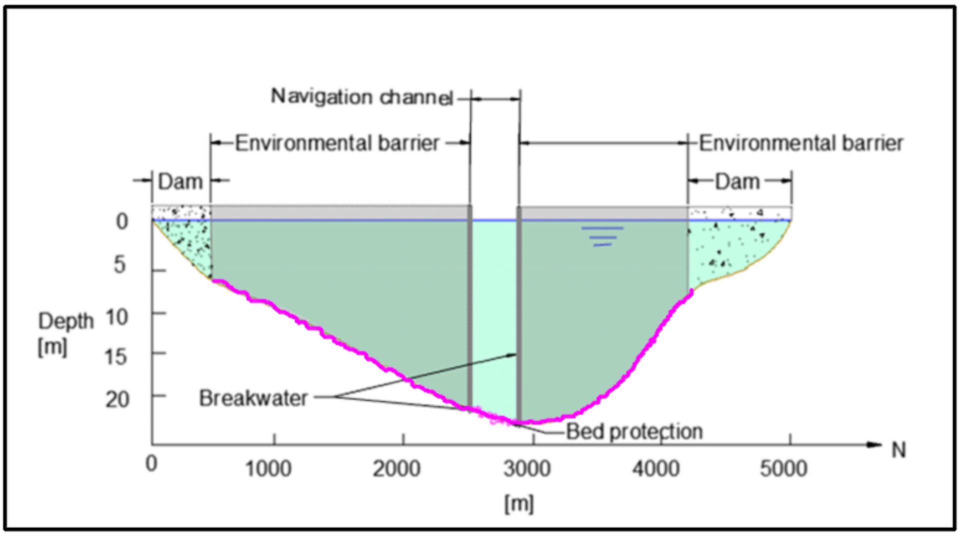


Figure 10- 2. Schematic front view of the 'open-option' in the South Channel (not to scale)

10.2 Scouring problem during critical conditions

Before thinking of protection works, one should have an idea of the possible erosion. Erosion occurs everywhere in nature where the bed consists of fine sediments(Schiereck, 2001). The disruptions in river cause changes in current acceleration or deceleration, which then in turn influence the local sediment transport. The scouring process is virtually unavoidable. The scour itself is not a problem, but its threat to the stability of a structure. Thus, the main idea of the protection is to keep the scour hole as far away as possible from the structure.

10.2.1 Velocity profile

In general, scour is a special case of sediment transport. The local velocity and the turbulence pattern determine the local erosion. During normal conditions, all the gates included the environmental sections are open, so the calculations will only focus on the critical conditions. Under critical conditions, it is easy to compute the depth-average velocity through the gap by using MathCAD. The maximum depth-average velocity is resulted to be 7.8m/s when the floodwater passes through opening. Assuming the Yangtze Estuary to be a rectangular basin (see Figure 10- 3), the depth-average velocities behind the gap in the centerline can be calculated by Delft3D, see Figure 10- 4.

It is clearly seen, the further into the estuary, the lower the current velocities. The computed velocities are depth averaged. The results should time a coefficient (around 0.9) to obtain the near-bed velocities(Schiereck, 2001). But which one should be used depends on the selected calculation method.

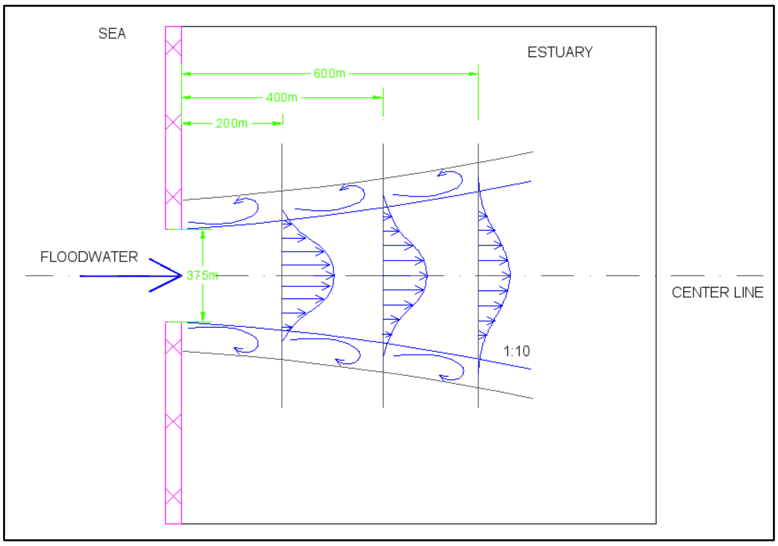


Figure 10- 3. Velocity profile and influence of turbulence behind gap

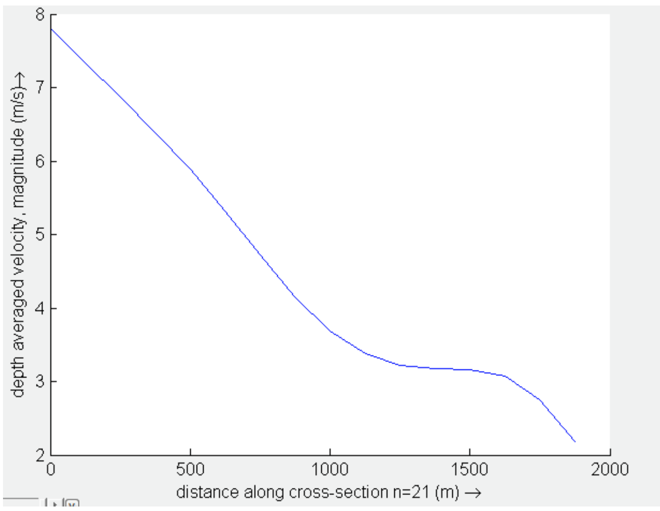


Figure 10- 4. Depth-average velocities behind the gap in the centerline

10.2.2 Scour without protection

Protection against scour is not always necessary, as scour as such is not the problem. Only if the stability of a structure is endangered, a protective method is required. In order to be able to decide whether measures are to be taken, insights should be taken into the degree of scour without protection. The equilibrium scour depth h_{se} can be calculated with(Schiereck, 2001):

$$h_{se} = h_0 \times \frac{0.5\alpha\bar{u} - \bar{u}_c}{\bar{u}_c} \qquad \text{Equation 10- 1}$$

Where,

- h_{se} is the equilibrium scour depth [m]
- h_0 is the average water depth [16.25 m]
- $\bar{u}_c$ is the critical velocity of be material [for this project $\bar{u}_c$ = 0.4 m/s]
- $\bar{u}$ is the local current velocity [m/s]
- α is the amplification factor due to turbulence [without protection $\alpha = 5$]

Instituting all the parameters, the equilibrium scour depth h_{se} equals 715m. Of course this is a too conservative value, because the high velocity up to 7.8m/s only occurs during a short period (~12 hours), the equilibrium state might not reach. Then the study on the scour development in time should be done. But from here it is obviously concluded, the bed protection is indeed required.

10.2.3 Case study of existing projects

Bed protection is required almost for all large hydraulic structures. Here two examples from the Netherlands are studied.

10.2.3.1 Maeslant barrier

During gate closure, the gates are lowered on top of concrete sills. High velocities occur between the gates and the concrete blocks, due to the large head difference across the barrier. The critical condition is closing the gate during a flood flow. In that case, the flow accelerates at the seaside while it decelerates at the riverside.

The original bed material consists of fine sand and silt. The first filter layer is sand ranging from 0.5 to 5 mm. Further away from the sill, the stone size in the top layer decreases. The other filter layers below the top layer are gravel and rocks with difference sizes, which are shown in Figure 10- 5.

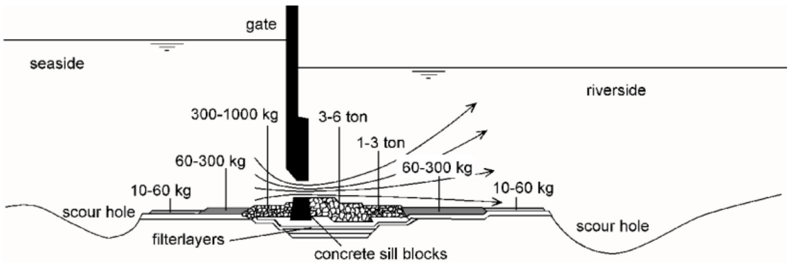


Figure 10- 5. Cross section bed protection of Maeslant barrier(Schiereck, 2001)

10.2.3.2 Eastern Scheldt barrier

For the Eastern Scheldt barrier, a large area has to be protected. According to the design criteria, the bottom near the gates should be protected, when large velocities occur through the gates while closing or when all gates are closed except one (the probability of one gate not being closed on time is rater high with 66 gates). This failure scenario is quite similar to the 'open-option' in this project: only one small opening exists while floodwater comes.

Compared with Maeslant barrier, the construction of loose rock protections is too time-consuming and expensive. Thus a fascine mattress is used in this project. But the construction of such a filter with fine material in flowing water of around 25 m depth drives a solution with a sophisticated geotextile mat.

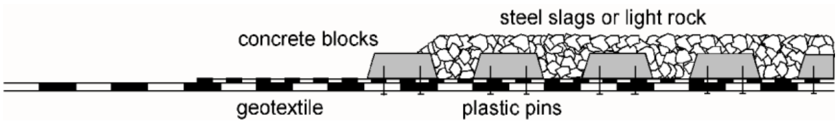


Figure 10- 6. Block mat of Eastern Scheldt barrier(Schiereck, 2001)

10.3 Scour protection design for critical condition

According to calculations in previous section, it is obvious the potential scours will threaten the stability of structures. The protection measures will be discussed in this part.

10.3.1 Design inputs

The first assumption is about the Yangtze Estuary. To simplify calculation, the estuary is modeled as a rectangular semi-closed basin, with 15km width and 50km length. The bed of the estuary consists of loosely packed sand with a diameter of 0.2 mm.

One of the most important factors influencing the design is the current velocity. It is assumed that the velocities keep constant every 50 m behind the opening, and only the maximum velocity located in the centerline dominates. The influence of turbulence is assumed to valid only in the area of 25m away to the streamline, see Figure 10- 7.

Table 10- 1. Velocity distribution behind opening during floods

Distance to the barrier(m)	Symbol	Depth-average velocity (m/s)
through opening	u0	7.80
0	u1	7.02
50	u2	6.85
200	u5	6.30
250	u6	6.21
300	u7	6.03
350	u8	5.95
400	u9	5.75
450	u10	5.58
500	u11	5.13
550	u12	4.95

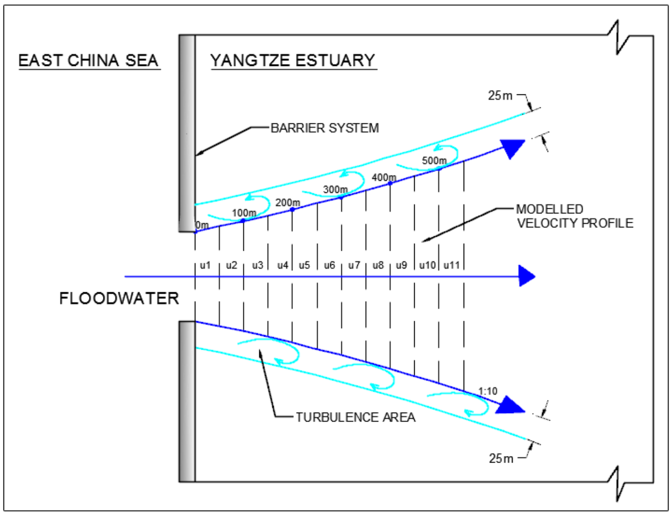


Figure 10- 7. Modeled discrete velocity profile

The critical condition is closing all the environmental gates, while leaving the navigational section open during floods. Take the Eastern Scheldt barrier as a reference, the protection length is assumed to be 400m. But this value can be modified more or less during the design process.

10.3.2 Protection methods

Scouring is a special case of erosion. If erosion occurs somewhere, generally some options are available:

- Do nothing.
  According to previous section. The scour depth just behind the opening can be up to hundred meters, which is of course unacceptable.
- Take away the cause of the problems
  For bed protection, the cause of erosion is turbulence, while removing the structures is not possible.
- Supply sediment.
  This seems to be a poor solution, since it cures nothing and it may have to go on forever. Whether it is feasible or not depends on the costs and risks. Since the sever scour hole is caused by floodwater, which only occurs twice or three times per year. But the amount of sediment is difficult to predict. Further treatment of the option should be part of the morphological studies and is beyond this study.
- Reduce the loads.
  Load reduction can be realized by streaming the outflow of floodwater, leading to less turbulence or lower velocity. Again the effects are part of morphological studies.
- Increase the strength.
  Increase of strength by a bottom protection is always the most feasible solution. Actually this is not a very flexible solution and it can sometimes cause more erosion in adjacent areas. But the main function of bottom protection is not to prevent scour but

to keep it at such a distance from the structure, that to minimize the risk of falling. This project will focus on this hard solution only.

This bed protection is aimed to protect the estuary side riverbed behind the barrier. The widely used system includes rocks, interlocked concrete blocks and box gabions. There are also lots of stability functions for various systems, one of which is the formula of Pilarczyk. This formula concerns the relationship between the element size and the related parameters of structure(Thorne, Abt, & Maynord, 1995):

$$D_{50} = \frac{\emptyset_{sc}}{\Delta} \frac{0.035}{\Psi_{cr}} k_h k_{sl}^{-1} k_t^2 \frac{u^2}{2g} \qquad \textit{Equation 10- 2}$$

Where,

- D_{50} is the size of protection element [m]
- $\emptyset_{sc}$ is the stability correction factor [-]
- Δ is the relative buoyant density of the protection element [-]
- Ψ_{cr} is the critical mobility parameter of the protection element [-]
- k_h is the velocity profile factor [-]
- k_{sl} is the side slope factor [-]
- k_t is the turbulence factor [-]
- u is the depth-averaged flow velocity [m/s]

The parameters above can be decided by using Rock Manual(Manual, 2007).

Table 10- 2. Parameters of Pilarczyk equation (Manual, 2007)

Characteristic size, D	• armourstone and rip-rap: $D = D_{n50} \cong 0.84 D_{50}$ (m) • box gabions and gabion mattresses: D = thickness of element (m) **NOTE:** The armourstone size is also determined by the need to have at least two layers of armourstone inside the gabion.
Relative buoyant density, Δ	• rip-rap and armourstone: $\Delta = \rho_r/\rho_w - 1$ • box gabions and gabion mattresses: $\Delta = (1 - n_v)(\rho_r/\rho_w - 1)$ where n_v = layer porosity $\cong$ 0.4 (-), ρ_r = apparent mass density of rock (kg/m³) and ρ_w = mass density of water (kg/m³)
Mobility parameter, ψ_{cr}	• rip-rap and armourstone: $\psi_{cr} = 0.035$ • box gabions and gabion mattresses: $\psi_{cr} = 0.070$ • rock fill in gabions: $\psi_{cr} < 0.100$
Stability factor, ϕ_{sc}	• exposed edges of gabions/stone mattresses: $\phi_{sc} = 1.0$ • exposed edges of rip-rap and armourstone: $\phi_{sc} = 1.5$ • continuous rock protection: $\phi_{sc} = 0.75$ • interlocked blocks and cabled blockmats: $\phi_{sc} = 0.5$
Turbulence factor, k_t	• normal turbulence level: $k_t^2 = 1.0$ • non-uniform flow, increased turbulence in outer bends: $k_t^2 = 1.5$ • non-uniform flow, sharp outer bends: $k_t^2 = 2.0$ • non-uniform flow, special cases: $k_t^2 > 2$ (see Equation 5.226)
Velocity profile factor, k_h	• **fully developed logarithmic velocity profile:** $k_h = 2/\left(\log^2(1+12h/k_s)\right)$ (5.221) where h = water depth (m) and k_s = roughness height (m); k_s = 1 to $3D_n$ for rip-rap and armourstone; for shallow rough flow ($h/D_n < 5$), $k_h \cong 1$ can be applied • **not fully developed velocity profile:** $k_h = \left(1 + h/D_n\right)^{-0.2}$ (5.222)
Side slope factor, k_{sl}	The side slope factor is defined as the product of two terms: a side slope term, k_d, and a longitudinal slope term, k_l: $k_{sl} = k_d\, k_l$ where $k_d = (1 - (\sin^2\alpha / \sin^2\phi))^{0.5}$ and $k_l = \sin(\phi - \beta)/(\sin\phi)$; α is the side slope angle (°), ϕ is the angle of repose of the armourstone (°) and β is the slope angle in the longitudinal direction (°), see also Section 5.2.1.3.

Keeping one 375m opening during floods will result in sudden flow acceleration. The high flow current and high turbulence will cause damage to the stability of adjacent structures. This part will give solutions to prevent failure, and the computation area is 46.5m*375m, see Figure 10- 8.

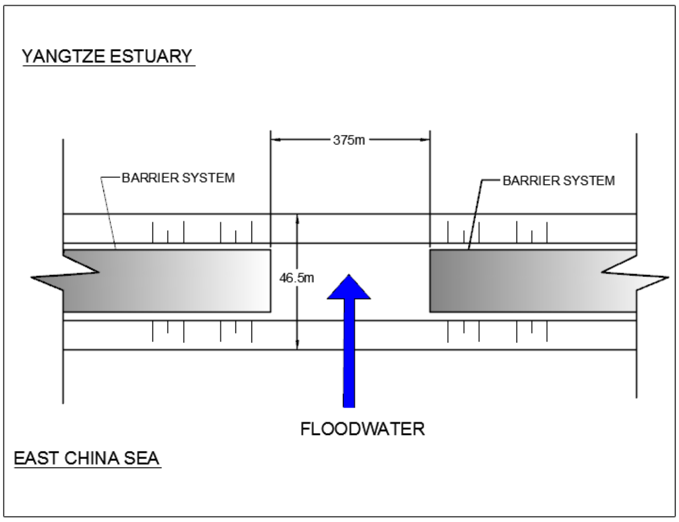


Figure 10- 8. Flow through opening during floods

11.2.1 Top layer

The velocities to be used in calculations, through the opening, are modelled with Delft3D. The maximum depth-average velocities are a little less than 7.8m/s. It can be assumed that the maximum velocity dominates the bottom protection design over the whole protection area.

Table 10- 3. Results from Pilarczyk equation in the area through opening

Element	D	unit	remarks
rock	2.30	m	D=Dn50
concrete block	0.63	m	thickness
box gabion	0.96	m	thickness

The results from the Pilarczyk Equation (Equation 10-2) are listed in Table 10- 3. According to experience, rock is most of time the best choice except the size is too large. According to the result, rock size of Dn50=2.3m is far away beyond the standard grading in EN13383, which would cause problem for construction.

Concrete blocks can be better alternative. Then one single layer interlocked concrete blocks (for example X-bloc) can be placed in the area with 0.63m thickness. This option seems attractive, however, interlocked concrete blocks are particularly designed to increase stability on slopes, which is not required for bed protection on flat bed.

Box gabion is more suitable for heavy current attack when the required stone size is limited. According to the result, the standard gabion box with size of 1m (width)*1m (height)*2m (length) can be used. In this project, the gabion structure needs impermeability and weight to counter uplift. To give these characteristics, the stone is grouted with mastic. Another advantage of grouted stone-filled gabions can give some protection to the weir mesh against abrasion and corrosion. But the wire still requires to be PVC-coated from an environmental point of view. In sizing gabions, the following additional requirements should be observed(May, Ackers, & Kirby, 2002):

- (minimum) stone size> 1.25*maximum spacing between wires;
- (maximum) stone size<2/3*height of box
- (minimum) height of box>0.15m

Considering the requirements and results from Pilarczyk equation, the recommended stone size for box gabion ranges from 0.4m to 0.6m. The spacing between wires should not exceed 0.32m.

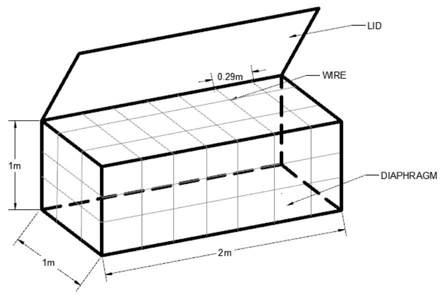


Figure 10- 9. Box gabion dimensions

11.2.2 Filter rule

In order to prevent migration of soil through the protection, the box gabions should be used in conjunction with geotextile filter layers.

The criteria for geotextile filter selection are in short as follows:

A **retention criterion** to ensure that the openings are small enough to prevent excessive migration of sands. This criteria can be expressed with equation:

$$O_{90} < 2d_{90B} \qquad \textit{Equation 10- 3}$$

Where,

- O_{90} is the nominal opening diameter of geotextile [m]
- d_{90B} is the size exceeded by 10% of the material of base layer [= 0.5mm]

A **permeability criterion** to ensure that the geotextile is permeable enough to allow water to pass through without impendence. A simple rule is that the permeability, k, of the geotextile should be more than 10 times larger than that of subsoil. The permeability of subsoil is around 0.002 1/s. The criterion reads,

k> 0.02 (1/s) *Equation 10- 4*

An **overall stability criterion** to ensure that the combination of filter layers stay tight. Because this geotextile will be built on flat bed, no requirement is set for internal friction.

The **survivability and durability criterion** to ensure the installation and the geotextile is functional during its lifetime. This criterion can be fulfilled with limiting the damping height of rocks. But in this case, the upper layer of geotextile will be box gabions. The durability criterion can be easily guaranteed, as due to tests after a century the geotextile can still work well.

According to all the requirements, the maximum opening in the mat (O_{90}) should be smaller than 1mm. Mats can be selected as a practical solution with $O_{90} = 0.5mm, k = 0.05 - 0.5\ (1/s)$. In this project, the filter gradient parallel to the bottom is equal to the water surface (in the order of magnitude of less than 0.01), so a simple geotextile is enough. The stone-grouted box gabions are attached to the geotextile with plastic pins. The box gabions sink the mats and keep the mats stable. After sinking, the mats will be covered with steel slags for more stability in turbulent flows. The dimensions of mats are 50*375m^2. The sinking of mats requires special equipment, but it is possible for this large project. The artificial island mentioned in Section 9.8.2 can be used as the construction pit. The mats will be constructed on the pit and the transported to certain locations and dropped into water.

10.3.3 Bottom protection behind opening

Through the gap, the velocities increase significantly. Behind the gap, the flow spreads over the whole width and depth of the water area, leading to lower velocities. But the original bed material is still too weak to sustain such velocities. The most widely used materials include rocks (rip-raps), concrete blocks, gabions and mattress. Installation of mattress underwater with depth around 20m is difficult and the construction cost would be high, thus the mattress will not be considered.

10.3.5.1 Top layer design

It is not cost-effective to construct the whole protection with the same element size, because the velocities decrease quickly behind the barrier. Further downstream of the flow, smaller elements can be applied, since the estuary is quite wide and the flow spreads and the intensity of the turbulence becomes much lower.

For economic reasons, it is decided to divide the whole protection area into three areas A1 (0-150m), A2 (150-350m) and A3 (350m-400m), more information is presented in Figure 10-10. Then the required protection elements can be computed.

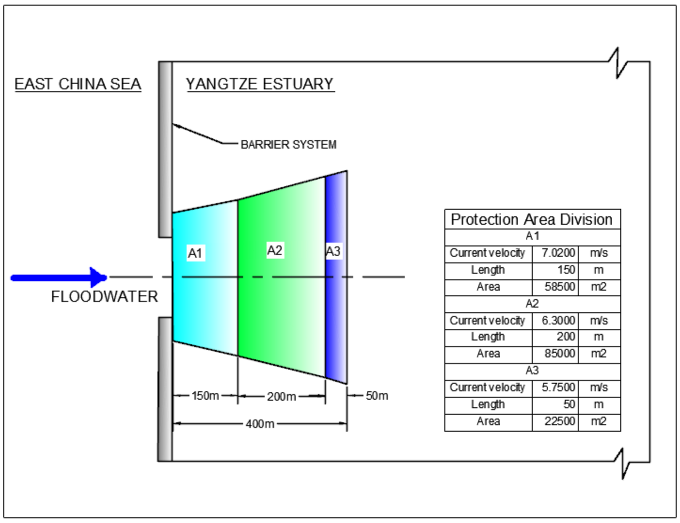


Figure 10- 10.Required protection area divisions

For this protection area behind the opening, all of the three options can be applied. But from the economical point of view, protection of rocks (rip-raps) is the most favorable due to its low labor requirements for its placement. Also, because the construction work will be done in flowing water, so it is recommended to use heavy rocks. The required rock sizes are summarized in Table 10- 4. The top layer thickness is calculated as $3D_{n50}$, to give more stability in flowing water during construction.

Table 10- 4. Selected element sizes for each area (unit: m)

Element	A1	A2	A3
ROCK	Dn50=1.71m	Dn50=1.03m	Dn50=0.70m
	6-10ton	1-3ton	300-1000kg
	Thickness 3.24m	Thickness 2.7m	Thickness 1.77m

10.3.5.2 Filter rule

The flow velocity and turbulence of flow can penetrate through the large stones of the bottom protection. Damping large stones directly on the sand bed can lead to erosion underneath. A geometrically closed granular filter will be designed to prevent erosion under top layer and for drainage to prevent pressure built-up.

Geometrically closed filters have to be designed within margins of stability and permeability. The design consists three parts:

Stability:

$$\frac{d_{15F}}{d_{85B}} < 5 \qquad \textit{Equation 10- 5}$$

Internal stability:

$$\frac{d_{60}}{d_{10}} < 10 \qquad \textit{Equation 10- 6}$$

Permeability:

$$\frac{d_{15F}}{d_{15B}} > 5 \qquad \textit{Equation 10- 7}$$

Applying geometrical filter rules, for the 6-10 ton rock a d_{15} of 156 cm is found. For stability, this leads to a maximum d_{85} for the next layer of 31.2 cm; for permeability, this leads to a minimum d_{15} for the next layer of 31.2 cm, corresponding with stones of 5-40 kg. For the construction and friction between layers a somewhat larger stone would be preferable, leading to 60-300 kg. From this layer, the same equations are used, leading to following layers with D_{50} of 8cm, 1.1 cm and 0.2cm. From the calculation, in total four filter layers are required for area A1. The last two layers are fine gravels mixed with sands. Each layer of 30 cm thickness is required. It is too time consuming and complex to sink such fine gravels. Thus, these two layers can be replaced with filter mats of 50 cm thick.

Repeating the same process, the filters for A2 and A3 can be calculated. The results are summarized in Table 10- 5 and Figure 10- 11.

Table 10- 5. Bottom protection behind opening in A1 A2 and A3

A1			
Bottom protection layers	D50 (cm)	Range	Thickness(cm)
Top layer	171.43	6-10ton	324
Filter layer 1	42.00	60-300kg	85.5
Filter layer 2	8.00	45-200mm	50
Filter mat	Thickness 50 cm		
Base layer	0.02	-	-
A2			
Bottom protection layers	D50 (cm)	Range	Thickness(cm)
Top layer	103.00	1-3ton	270
Filter layer 1	20.00	10-60kg	30
Filter layer 2	7.00	90-250mm	30
Filter mat	Thickness 25 cm		
Base layer	0.02	-	-
A3			
Bottom protection layers	D50 (cm)	Range	Thickness(cm)
Top layer	70.24	300-1000kg	177
Filter layer 1	20.00	10-60kg	60
Filter layer 2	7.00	90-250mm	30
Filter mat	Thickness 25 cm		
Base layer	0.02	-	-

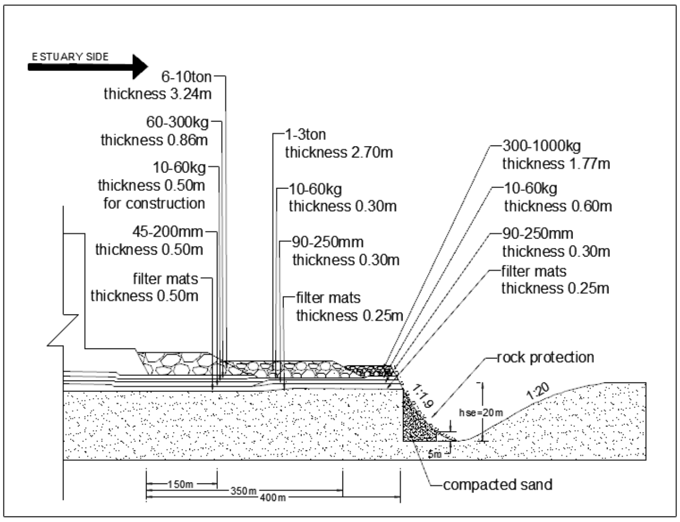


Figure 10- 11. Designed bottom protection in the estuary side behind opening of open option

10.3.4 Protection stability

In reality, scour is impossible to be avoided. The key solution lies in keep the scouring hole as far away from the structure as possible. Erosion behind the protection could also lead to instability of structures if the scouring hole is too deep and extends to the protected element. It is dependent on both the scouring depth and upstream slope.

10.3.5.1 Scour depth behind protection

In literature, two main approaches can be distinguished: one is to study the final equilibrium scour depth and the other is to study the development in time. In this project, the critical situation lasts only for several hours (flood period ~12 hours). Thus, both of the two methods will be discussed. Then the stability of the protection will be checked. All the equations used in this part are based on knowledge from the book '*Introduction to bed bank and shoreline protection*'.

Scour development with time

Many tests have been performed to find the scour development with time. The following expression can be used for the preliminary analysis clear-water scour behind bed protection:

$$h_s(t) = \frac{(\alpha\bar{u}-\bar{u}_c)^{1.7} h_0^{0.2}}{10\Delta^{0.7}} t^{0.4} \qquad \textit{Equation 10- 8}$$

Where,

- h_s is the scour depth [m]
- h_0 is the average water depth [m]
- $\bar{u}$ is the local current velocity [m/s]
- $\bar{u}_c$ is the critical velocity of bed material [m/s]
- t is the time [hour]

- α is the amplification factor due to turbulence, which can be determined as following paragraph [-]

For longer bed protection L > $5h_0$, α-value can be derived using following relationship:

$$\alpha\left(\frac{L}{h_0}\right) = 1.5 + (1.57\alpha_{10} - 2.35)e^{(-0.045L/h_0)}$$ *Equation 10- 9*

Where,

- L is the protection length [m]
- α_{10} is the α-value for L/h_0=10 ($\alpha_{10} = 4$, which can be determined from Figure 10- 12 with b/B=0.95) [-]

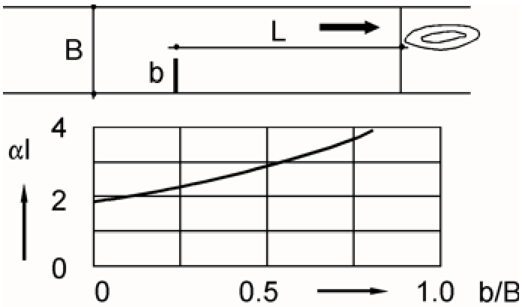


Figure 10- 12. Values for α as a function of horizontal constriction for L/h_0=10

In combinations with boundary conditions, there is no problem computing the scouring depth as a function with time. But the design condition is a time-varying condition with tides, so a tidal average of the scouring process will be performed here. The process is also shown in Figure 10- 13. First, $\alpha\bar{u} - \bar{u}_c$ is determined for every half hour, raised to the power 1.7 and the average is calculated over the whole tidal range. This results in a value of 50.5. This is the value to be used in Equation 10-3 to replace$(\alpha\bar{u} - \bar{u}_c)^{1.7}$. The result is shown in Figure 10- 14. The critical situation lasts for less than 24 hours; during the floods the scour depth would not exceed 20m.

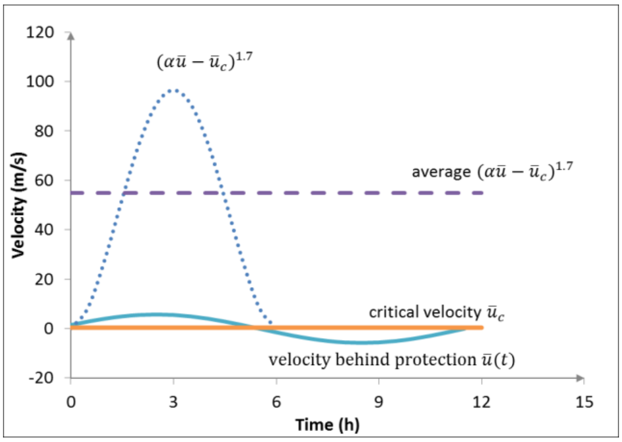


Figure 10- 13.Relevant scouring parameter

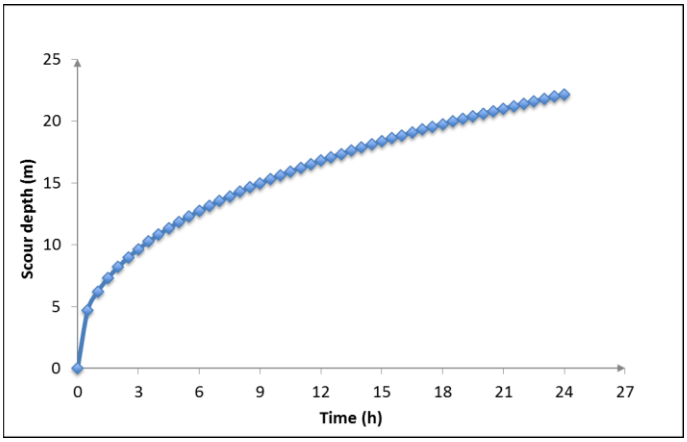


Figure 10- 14. Scour development with time within one day

Equilibrium scour

The equilibrium scour depth can be derived from:

$$\frac{h_{se}}{h_0} = \frac{0.5\alpha\bar{u} - \bar{u}_c}{\bar{u}_c} \qquad \textit{Equation 10- 10}$$

Where h_{se} is the equilibrium scour depth, the other parameters are the same as those in Equation 10-3. But the dominant flow velocity is the average velocity occurring just behind the protection.

One to be noted is that, the equilibrium only valid for the equilibrium state. The design condition is when the floods pass through the opening, which only lasts for several hours. Thus, the value of equilibrium scour depth is meaningless here.

10.3.5.2 Protection length

The question now is to check whether the 400 long protection is enough. To judge it, it is necessary to know the upstream slope β. From the systematic research, Hoffmans, 1993, derived the relation:

$$\beta = arcsin(3 \times 10^{-4} \frac{u_0^2}{\Delta g d_{50}} + (0.11 + 0.75 r_0) f_c$$

Equation 10- 11

Where,

- β is the upstream slope of scour hole [-]
- r_0 is the turbulence intensity [=0.1]
- f_c is friction coefficient, $f_c = \frac{C}{40}$ $[\sqrt{m}/s]$

Substituting all the parameters into Equation 10-6, the slope results to be 1.9. At the beginning of this chapter, it is assumed that the sand is loosely packed. That means, the danger of flow slides exists, with very gentle slope (order of magnitude 1:15). With a slope of the scour hole itself of 1:1.9, the flow slides would not damage the structures (20*15 < 400+20*1.9). If one is still afraid of the bottom protection being too short, the solutions can be to lengthen the protection or dump slags or gravels on the scour slope adjacent to apron.

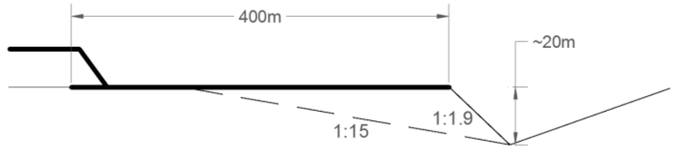


Figure 10- 15. Scour hole behind bed protection

11. COMPARISON BETWEEN 'GATE OPTION' & 'OPEN OPTION'

The city of Shanghai, the most important city in the East China, is relatively low-lying. Therefore, it is threaten by floods coming from seaside, along with potential river floods from Yangtze River. In and around the city, a number of rivers and artificial canals are present. Therefore, floods can enter the city rapidly. To mediate this situation, several solutions are proposed in Chapter 5. Then it is decided that a barrier system closing the Yangtze Estuary is the best option.

One main part of the barrier system is the navigational section, which is kept open during normal conditions to allow free shipping. The regular solution is to have a movable gate in this navigational section, which means "open in normal conditions but closed during storm conditions". But the gate is always the main part of the total construction cost. It is attractive to consider a permanent opening without gate, which refers to the 'open option' in this report. Both options have been discussed in detail in Chapter 9 and 10. This part will go further for a comparison of these two options, mainly focus on the general cost estimations.

The cost of barrier system is estimated on a pre-feasibility level. Unit price used in this report stems from both the Dutch[19] and Chinese[20] market for the 2014 price level.

11.1 Review of both options

Firstly, a review of the two options will be discussed in this section. A general overview of the barrier system will be summarized, including the navigational sections, environmental sections and the alignments with other structures.

11.1.1 Gate option

The Yangtze River is the largest river in China. Shanghai locates at the river month to the sea. One option is to build a barge gate in the South Chanel, see Chapter 9. The following map provides an overview of 'gate option' of barrier system in the Yangtze Estuary.

[19] Dutch price unit of Euro, reference to a technic report by ARCADIS.
[20] Chinese price unit of Yuan, reference to Alibaba Press http://www.alibaba.com/ and *based on a cost-estimation report of newly- built breakwater along East China Sea coastline.*

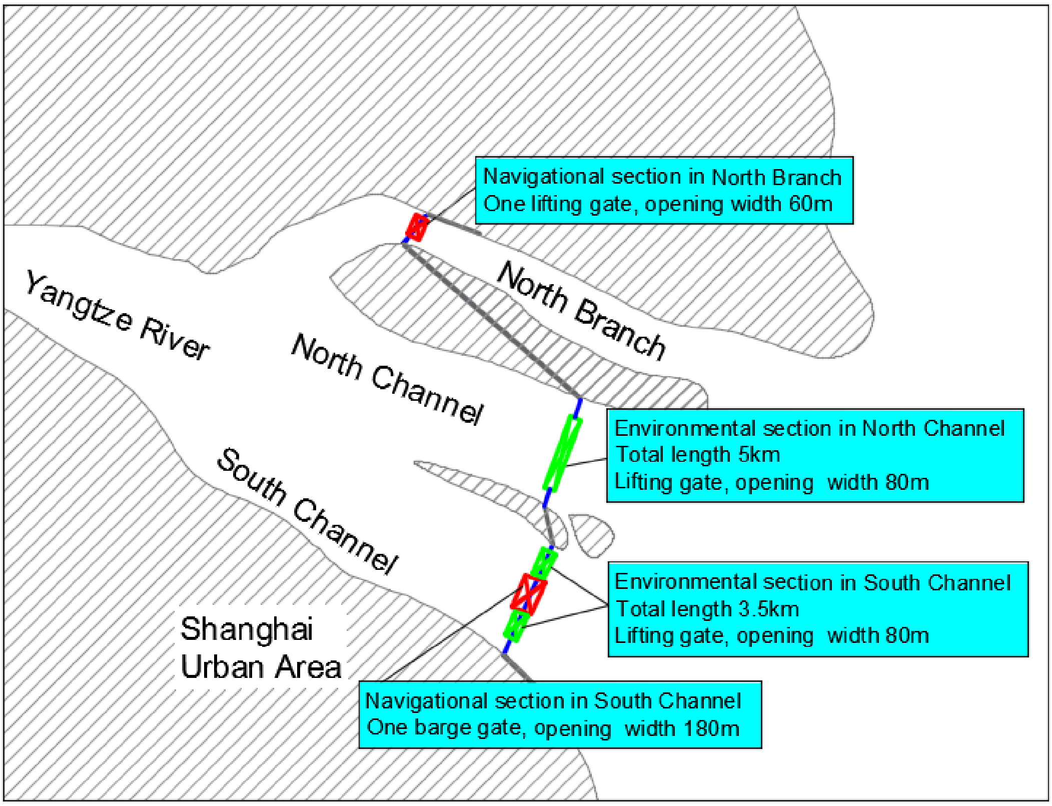


Figure 11- 1. Overview of gate option of barrier system in the Yangtze Estuary

Based on a forecast of high water level in East China Sea, all the gates are closed. All navigation is blocked as long as the barrier is closed. A barrier consists of numerous objects, which are needed to close off waterway. The main objects of this barrier system which are indicated in Figure 11- 2 are:

- the Gates
  - The gates are the water retaining element of the barrier. The gates withstand the storm surge. The gates transfer hydraulic forces to the substructures.
  - Two types of gates are required for this system. The lifting gates in the North Channel and the South Channel are used as environment gates for discharge of water flow. The horizontal sliding gate in the North Branch and the barge gate in the South Channel are designed for navigation purpose.
- the upstream bed protection
  - The upstream bed protection defends the bottom erosion at the upstream side of the structure. A bed protection is placed along the whole ‘wet’ structure, which withstands the flow and protects the riverbed form. The barrier could become unstable and collapse when the scour holes become too large in the vicinity of the structures.
- the downstream bed protection
  - The downstream bed protection defends the bottom erosion at the downstream side of the structure. A bed protection is placed along the whole ‘wet’ structure,

which withstands the flow and protects the riverbed form. The barrier could become unstable and collapse when the scour holes become too large in the vicinity of the structures.

- civil works
  - Extra gate recess is required for barge gate. An artificial island is designed as building pit and also serves as gate recess.
- seepage cutoff wall/sheet pilling
  - The sheet piling elongates the seepage length underneath the structure to prevent piping.
- supporting structures
  - The supporting structures must provide a stable support for the barge gate.
  - The abutments and foundation supports the barge gate in closed position.
  - The piers and abutments support the lifting gate, bulkheads, gate operation machinery, etc. and transfer the loads to foundation.

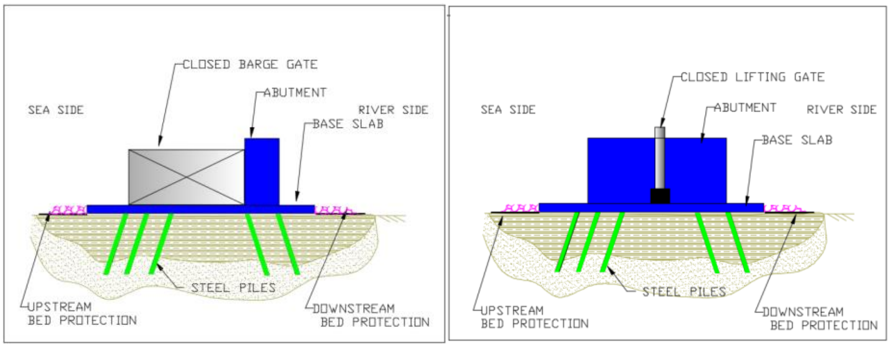


Figure 11- 2. Main gate objects (lift: barge gate; right: lifting gate)

11.1.2 Open option

Open-closable gate is the main part of the total construction cost. It is interesting to consider reducing the number of gates, especially the navigable gate. Thus, an option as 'open option' is proposed in Chapter 10.

The following map provides an overview of 'open option' of barrier system in the Yangtze Estuary.

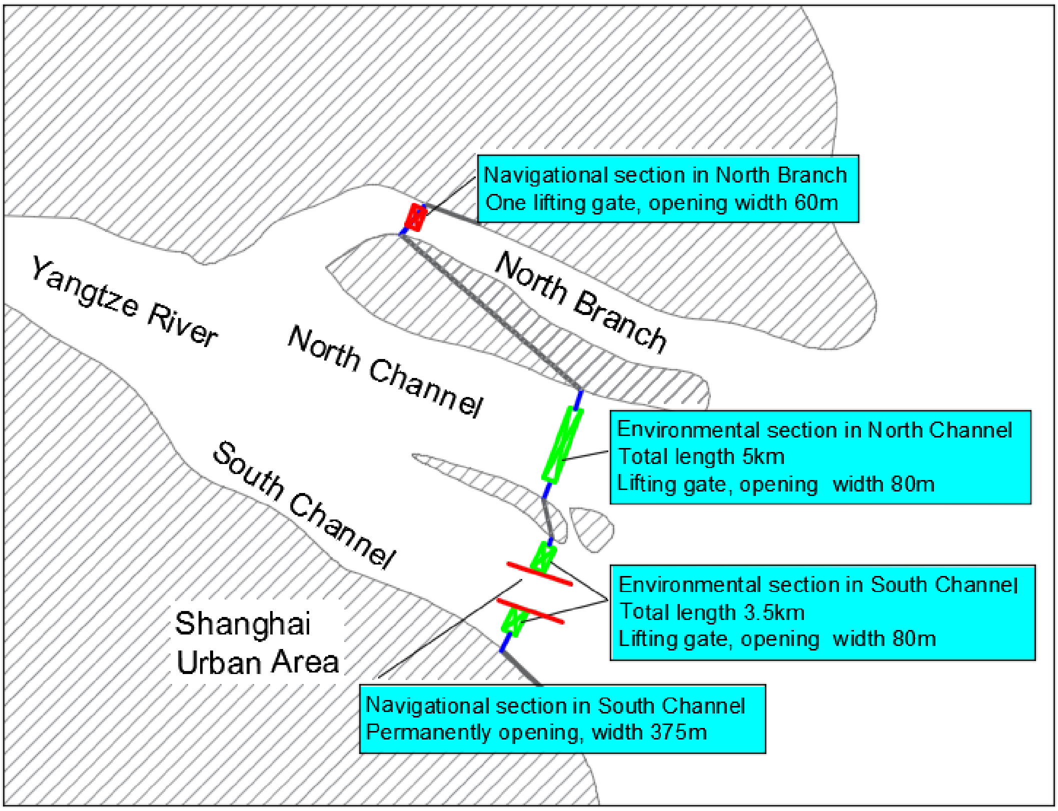


Figure 11- 3. Overview of open option of barrier system in the Yangtze Estuary

Based on a forecast of high water level in East China Sea, all the gates are closed. All navigation is blocked as long as the barrier is closed. A barrier consists of numerous objects, which are needed to close off waterway. The main objects of this barrier system which are indicated in Figure 11- 4 are:

- the Gates
  - The gates are the water retaining element of the barrier. The gates withstand the storm surge. The gates transfer hydraulic forces to the substructures.
  - Two types of gates are required for this system. The lifting gates in the North Channel and the South Channel are used as environment gates for discharge of water flow. The horizontal sliding gate in the North Branch is designed for navigation purpose.
- the permanent opening
  - A permanent opening is placed at the same location as the barge gate in the gate option. This opening is designed to keep opening during all its lifetime. The water level inside the estuary must be within the safety level to prevent flooding.
- the upstream bed protection
  - The upstream bed protection defends the bottom erosion at the upstream side of the structure. A bed protection is placed along the whole ‘wet’ structure, which withstands the flow and protects the riverbed form. The barrier could become

unstable and collapse when the scour holes become too large in the vicinity of the structures.

- the downstream bed protection
  - The downstream bed protection defends the bottom erosion at the downstream side of the structure.
  - The barrier could become unstable and collapse when the scour holes become too large in the vicinity of the structures.
  - A bed protection is placed along the whole 'wet' structure, but varies in different location. Directly behind the opening, the bed protection should withstand the high current velocities and turbulent water. Other parts or bed protection are the same as those in the gate option.
- seepage cutoff wall/sheet pilling
  - The sheet piling elongates the seepage length underneath the structure to prevent piping.
- supporting structures
  - The supporting structures must provide a stable support for the gates.
  - The piers and abutments support the lifting gate, bulkheads, gate operation machinery, etc. and transfer the loads to foundation.

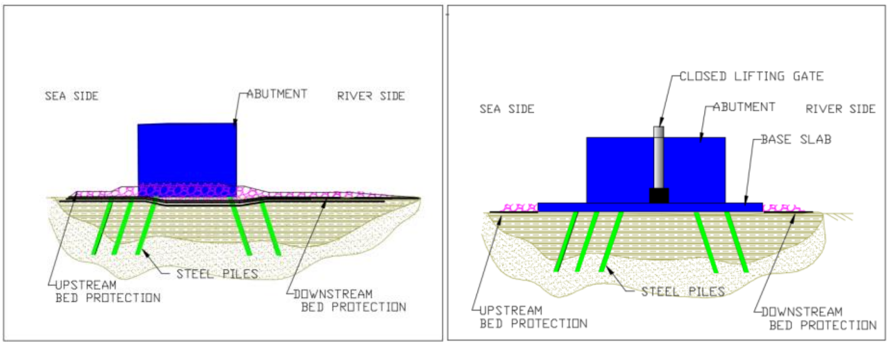


Figure 11- 4. Main gate objects (lift: opening in South Channel; right: lifting gate)

11.2 Cost estimation of two option

The costs for a pre-feasibility level are based on the estimated costs of the main objects listed in the previous section. The substantiation of costs per main objects is presented in the sequential paragraphs.

11.2.1 Assumptions for estimation

The level of detail of the pre-feasibility design is very low; therefore assumptions of main elements for the cost estimation have been made:

- Gates
  - No details of lifting gates and sliding gate;
  - Hydraulic equipment in included as 10% of estimated costs;

  - It is assumed the gates will be transported by barge and installed by a floating crane
- Bed protection:
  - No excavation of subsoil;
  - Use gravel, rocks, concrete and fascine mattress.
- Seepage cut-off screen:
  - No complicated connection to the surrounding;
  - Installation by a floating barge.
- Super- and sub-structures:
  - Costs of finishing works, sills, and soil improvement are not estimated, but are included as 20% of the estimated cost.
- Foundation:
  - Steel tubular sheet piles are used, with diameter of 1 meter;
  - Foundation piles will be driven to WD-65m into the bearable bed layer;
  - A grid of 4m*4m is used.
- Construction aspects:
  - The artificial islands serve as construction pit;
  - The barrier should be realized in two phases in order to maintain an open connection between the upstream and downstream.
- The costs are excluded:
  - Additional costs like research, mitigation etc.
  - Costs for cables and cranes
  - Costs for surroundings
  - Cost for architectures
  - Realizing costs due to phasing
  - Uncertainty allowance

11.2.2 Cost estimation of gate option

The cost of barrier system is estimated on a pre-feasibility level. Unit price used in this report stems from both the Dutch and Chinese markets for the 2014 price level. The detailed design of gate option is described in Chapter 9.

11.2.2.1 Gates

The dimensions of gates are summarized in

Table 11- 1.

Table 11- 1. Dimensions of gates in Yangtze Estuary of gate option

	Navigational gates		Environmental gates	
	Parameter	Value	Parameter	Value
South Channel	Gate type	Barge gate	Gate type	Vertically lifting gate
	Material	Prestressed concrete	Material	Steel
	Opening width	180m	Opening width	80m
	Number of openings	1	Number of openings	44
	Top level closed gate	WD+5.1m	Top level closed gate	WD+5.1m
	Sill level	WD-16.5m	Sill level	WD-12.5m
	Gate height	21.6m	Gate height	17.6m
North Channel			Gate type	Vertically lifting gate
			Material	Steel
			Opening width	80m
			Number of openings	63
			Top level closed gate	WD+5.1m
			Sill level	WD-10.5m
			Gate height	15.6m
North branch	Gate type	Horizontal sliding gate		
	Material	Concrete		
	Opening width	60m		
	Number of openings	1		
	Top level closed gate	WD+4m		
	Sill level	WD-5m		
	Gate height	9m		

The costs of gates depend on the weight of the gate, unit price per kg, the erection method and the hydraulics, which are necessary for operating the gates. The barge gate is made of prestressed concrete, thus the costs mainly lies on the materials costs. No detailed design of other gates is presented, so a reference is taken from a cost estimation report from ARCADIS. Costs of erection highly depend on the available equipment and the season. Costs used for the estimation are based on an installation-method. A barge transports the gates and then a floating crane will hoists the gate into position. Table 11- 2 represents the break-down of the estimated costs of gates.

Table 11- 2. Cost estimation of the gates of 'gate option'

Objects	Specification	Unit	Dimension	Unit cost		Total costs	
Gate type 1		1	Amount	€ 19,145,290	¥77,581,160	€ 19,145,290	¥ 77,581,160
Gate	Steel 53.7 ton	53.7	Metric ton	€ 500	¥ 2,000	€ 26,850	¥ 107,400
	Concrete 58176 m3	25983	m3	€ 680	¥ 2,720	€ 17,668,440	¥ 70,673,760
Barge		1	Amount	€ 100,000	¥ 400,000	€ 100,000	¥ 400,000
Crane		1	Amount	€ 200,000	¥ 800,000	€ 200,000	¥ 800,000
Supports and personnel		1	Amount	€ 50,000	¥ 200,000	€ 50,000	¥ 200,000
Hydraulics		1	Amount	€ 1,000,000	¥ 5,000,000	€ 1,000,000	¥ 5,000,000
Operation& co	10% of hydraulics	1	Amount	€ 100,000	¥ 400,000	€ 100,000	¥ 400,000
Total direct costs						€ 19,145,290	¥ 77,581,160
Total realisation costs excl. VAT						€ 19,145,290	¥ 77,581,160
Gate type 2		44	Amount	€ 7,858,000	¥31,432,000	€ 345,752,000	¥ 1,383,008,000
Gate	1183 tons	1183	Metric ton	€ 6,000	¥ 24,000	€ 7,098,000	¥ 28,392,000
Barge		1	Amount	€ 50,000	¥ 200,000	€ 50,000	¥ 200,000
Crane		1	Amount	€ 50,000	¥ 200,000	€ 50,000	¥ 200,000
Hydraulics		1	Amount	€ 600,000	¥ 2,400,000	€ 600,000	¥ 2,400,000
Operation& co	10% of hydraulics	1	Amount	€ 60,000	¥ 240,000	€ 60,000	¥ 240,000
Total direct costs						€ 7,858,000	¥ 31,432,000
Total realisation costs excl. VAT						€ 7,858,000	¥ 31,432,000
Gate type 3		63	Amount	€ 7,054,000	¥28,216,000	€ 444,402,000	¥ 1,777,608,000
Gate	1049 tons	1049	Metric ton	€ 6,000	¥ 24,000	€ 6,294,000	¥ 25,176,000
Barge		1	Amount	€ 50,000	¥ 200,000	€ 50,000	¥ 200,000
Crane		1	Amount	€ 50,000	¥ 200,000	€ 50,000	¥ 200,000
Supports and personnel		1	Amount	-		-	
Hydraulics		1	Amount	€ 600,000	¥ 2,400,000	€ 600,000	¥ 2,400,000
Operation& co	10% of hydraulics	1	Amount	€ 60,000	¥ 240,000	€ 60,000	¥ 240,000
Total direct costs						€ 7,054,000	¥ 28,216,000
Total realisation costs excl. VAT						€ 7,054,000	¥ 28,216,000
Gate type 4		1	Amount	€ 1,770,000	¥ 7,080,000	€ 1,770,000	¥ 7,080,000
Gate	250 tons	250	Metric ton	€ 6,000	¥ 24,000	€ 1,500,000	¥ 6,000,000
Barge		1	Amount	€ 25,000	¥ 100,000	€ 25,000	¥ 100,000
Crane		1	Amount	€ 25,000	¥ 100,000	€ 25,000	¥ 100,000
Supports and personnel		1	Amount	-	-	-	-
Hydraulics		1	Amount	€ 200,000	¥ 800,000	€ 200,000	¥ 800,000
Operation& co	10% of hydraulics	1	Amount	€ 20,000	¥ 80,000	€ 20,000	¥ 80,000
Total direct costs						€ 1,770,000	¥ 7,080,000
Total realisation costs excl. VAT						€ 1,770,000	¥ 7,080,000
Total costs of gates excl. VAT						€ 811,069,290	¥ 3,245,277,160

11.2.2.2 Upstream bed protection

The length of bed protection is estimated in Section 9.11. The thickness of each layer is represented in

Table 11- 3. The resulting estimated costs of the upstream bed protection are presented in Table 11- 4.

Table 11- 3. Dimensions of upstream bed protection of "gate option

Protection area	Parameter	Value	Unit
	Total rocks along bar	8,630,400	m3
	Total filter mats alon	1,087,500	m3
Protection area B1	Width	200	m
	Protection area	3,000,000	m2
	Rock 1-3 ton	Thickness 2.2	m
	Rock 60-300 kg	Thickness 0.86	m
	Rock 10-60 kg	Thickness 0.6	m
	Rock 90-250 mm	Thickness 0.3	m
	Filter mats	Thickness 0.5	m
Protection area B2	Width(m)	100	m
	Protection area (m2)	1,500,000	m2
	Rock 300-1000 kg	Thickness 1.1	m
	Rock 10-60 kg	Thickness 0.6	m
	Rock 90-250 mm	Thickness 0.3	m
	Filter mats	Thickness 0.25	m

Table 11- 4. Cost estimation of the upstream bed protection of 'gate option'

Objects	Unit	Dimens	Unit	Dimension	Unit cost		Total costs	
Rock 1-3 ton	3,828,000	m3	10,144,200	tons	€ 30	¥ 200	€ 304,326,000	¥ 2,028,840,000
Rock 300-1000 kg	957,000	m3	2,536,050	tons	€ 30	¥ 200	€ 76,081,500	¥ 507,210,000
Rock 60-300kg	1,496,400	m3	3,965,460	tons	€ 30	¥ 200	€ 118,963,800	¥ 793,092,000
Rock 10-60 kg	1,566,000	m3	4,149,900	tons	€ 30	¥ 200	€ 124,497,000	¥ 829,980,000
Rock 90-250 mm	783,000	m3	2,074,950	tons	€ 30	¥ 200	€ 62,248,500	¥ 414,990,000
Filter mats 0.25m th	-	-	1,740,000	m2	€ 20	¥ 150	€ 34,800,000	¥ 261,000,000
Filter mats 0.5 m thi	-	-	870,000	m2	€ 20	¥ 150	€ 17,400,000	¥ 130,500,000
Total direct costs excl. VAT							€ 738,316,800	¥4,965,612,000

11.2.2.3 Downstream bed protection

In this gate option, the water depth at the bed protection, the river width and the thickness of the bed protection are equal to the upstream bed protection. Therefore, the estimated costs of downstream bed protection are equal to the estimated costs of upstream bed protection.

Table 11- 5. Cost estimation of the downstream bed protection of 'gate option'

Objects	Unit	Dimens	Unit	Dimension	Unit cost		Total costs	
Rock 1-3 ton	3,828,000	m3	10,144,200	tons	€ 30	¥ 200	€ 304,326,000	¥ 2,028,840,000
Rock 300-1000 kg	957,000	m3	2,536,050	tons	€ 30	¥ 200	€ 76,081,500	¥ 507,210,000
Rock 60-300kg	1,496,400	m3	3,965,460	tons	€ 30	¥ 200	€ 118,963,800	¥ 793,092,000
Rock 10-60 kg	1,566,000	m3	4,149,900	tons	€ 30	¥ 200	€ 124,497,000	¥ 829,980,000
Rock 90-250 mm	783,000	m3	2,074,950	tons	€ 30	¥ 200	€ 62,248,500	¥ 414,990,000
Filter mats 0.25m th	-	-	1,740,000	m2	€ 20	¥ 150	€ 34,800,000	¥ 261,000,000
Filter mats 0.5 m thi	-	-	870,000	m2	€ 20	¥ 150	€ 17,400,000	¥ 130,500,000
Total direct costs excl. VAT							€ 738,316,800	¥4,965,612,000

11.2.2.4 *Seepage cut off*

The seepage length is determined by using Bligh and Lane equations for the barge gate. The head over the lifting gates and soil conditions are comparable to the head and soil conditions of the barge gate. So, the resulting seepage length determined by using Bligh and Lane are applied for the lifting gates.

The vertical and horizontal seepage lengths determine the area of seepage screens. The section is determined in Section 9.12. The estimated costs of sheet piles are presented in

Table 11- 6.

Table 11- 6. Cost estimation of the seepage cut off of 'gate option'

Objects	Specification	Unit	Dimension	Unit cost		Total costs	
seepage cut off screen		1	Amount	€ 100,947,000	¥168,245,000	€ 100,947,000	¥ 168,245,000
Section 1	180*38.5 m2	6,930	m2			-	-
Section 2	80*(44+63)*38.5 m2	329,560	m2			-	-
Total area of sheet pile		336,490	m2	€ 300	¥ 500	€ 100,947,000	¥ 168,245,000
Total direct cost						€ 100,947,000	¥ 168,245,000
Total direct costs excl. VAT						€ 100,947,000	¥ 168,245,000

11.2.2.5 *Foundation*

The barrier has to be founded so as to transport loads to subsoil. It is determined in Section 9.10, the barrier is founded on piles. Steel tubular sheet piles are used, with diameter of 1 meter. Foundation piles will be driven to WD-65m into the bearable bed layer.

Table 11- 7. Cost estimation of foundation of 'gate option'

Objects	Specification	Unit	Dimension	Unit cost		Total costs	
Foundation		1	Amount	€ 51,550,800	¥303,240,000	€ 51,550,800	¥ 303,240,000
Foundtion type 1							
Tubular steel piles,d=1000mm	Slab 48*180 m2	8640	m2				
Grid	4*4 m2	540	Amount				
Pile length	59.5m per pile	32130	m	€ 85	¥ 500	€ 2,731,050	¥ 16,065,000
Foundtion type 2							
Tubular steel piles,d=1000mm	Slab 44*(20*80) m2	70400	m2				
Grid	4*4 m2	4400	Amount				
Pile length	52.5m per pile	231000	m	€ 85	¥ 500	€ 19,635,000	¥ 115,500,000
Foundtion type 3							
Tubular steel piles,d=1000mm	Slab 63*(20*80) m2	100800	m2				
Grid	4*4 m2	6300	Amount				
Pile length	54.5m per pile	343350	m	€ 85	¥ 500	€ 29,184,750	¥ 171,675,000
Total direct costs excl. VAT						€51,550,800	¥303,240,000

11.2.2.6 *Civil works*

The estimated costs of supporting structures are related to the amount of concrete used. The determination of costs of connecting levees is based on levee dimensions. An estimate of such civil works is presented in Table 11- 8.

Table 11- 8. Cost estimation of civil works of 'gate option'

Objects	Specification	Unit	Dimension	Unit cost		Total costs	
Supporting structures							
Abutments	23*6*5 m3	1380	m3				
Base slab		179840	m3				
Total volumn of concrete		181220	m3	€ 680	¥ 2,720	€ 123,229,600	¥ 492,918,400
Levees							
Levee along banks		20	km				
Connecting levee		68.32	km				
Total volumn of concrete		78.32	km	€ 5,000,000	¥ 20,000,000	€ 391,600,000	¥ 1,566,400,000
Total direct costs excl. VAT						€ 514,829,600	¥ 2,059,318,400

11.2.2.7 Construction pit

A construction pit has to be created for the barrier realization. It is decided to have artificial islands at the navigational section. The island can also serve as the gate recess when the barge gate is at rest; and it can also be used as approaching breakwaters for vessels. The island should have the minimum area of 200*50 m^2. The unit cost is roughly estimated as 500,000 euros per island.

Table 11- 9. Cost estimation of construction pit of 'gate option'

Objects	Specification	Unit	Dimension	Unit cost		Total costs	
Construction pit		2	Amount	€ 500,000	¥ 3,500,000	€ 1,000,000	¥ 14,000,000
Total direct cost				€ 500,000	¥ 3,500,000	€ 1,000,000	¥ 14,000,000
Total direct costs excl. VAT						€ 1,000,000	¥ 14,000,000

11.2.3 Cost estimation of open option

The cost of barrier system is estimated on a pre-feasibility level. Unit price used in this report stems from both the Dutch and Chinese markets for the 2014 price level. The detailed design of gate option is described in Chapter 10.

11.2.3.1 Gates

The dimensions of gates are summarized in Table 11- 10.

Table 11- 10. Dimensions of gates in Yangtze Estuary of open option

	Navigational gates		Environmental gates	
	Parameter	Value	Parameter	Value
South Channel	No gate		Gate type 5	Vertically lifting gate
	Permanently opened during all conditions		Material	Steel
	Opening width	375m	Opening width	80m
	Number of opening	1	Number of openings	42
			Top level closed gate	WD+7.0 m
			Sill level	WD-12.5m
			Gate height	19.5 m
North Channel			Gate type 3	Vertically lifting gate
			Material	Steel
			Opening width	80m
			Number of openings	63
			Top level closed gate	WD+5.1m
			Sill level	WD-10.5m
			Gate height	15.6m
North branch	Gate type 4	Horizontal sliding gate		
	Material	Concrete		
	Opening width	60m		
	Number of opening	1		
	Top level closed ga	WD+4m		
	Sill level	WD-5m		
	Gate height	9m		

The costs of gates depend on the weight of the gate, unit price per kg, the erection method and the hydraulics, which are necessary for operating the gates. No detailed design of lifting gates and sliding gate is presented, so a reference is taken from a cost estimation report from ARCADIS. Costs of erection highly depend on the available equipment and the season. Costs used for the estimation are based on an installation-method. A barge transports the gates and then a floating crane will hoists the gate into position. Table 11- 2 represents the breakdown of the estimated costs of gates.

Table 11- 11. Cost estimation of the gates of 'open option'

Objects	Specification	Unit	Dimension	Unit cost		Total costs	
Gate type 5		42	Amount	€ 7,858,000	¥ 31,432,000	€ 330,036,000	¥ 1,320,144,000
Gate	1183 tons	1183	Metric ton	€ 6,000	¥ 24,000	€ 7,098,000	¥ 28,392,000
Barge		1	Amount	€ 50,000	¥ 200,000	€ 50,000	¥ 200,000
Crane		1	Amount	€ 50,000	¥ 200,000	€ 50,000	¥ 200,000
Hydraulics		1	Amount	€ 600,000	¥ 2,400,000	€ 600,000	¥ 2,400,000
Operation& control	10% of hydraulics	1	Amount	€ 60,000	¥ 240,000	€ 60,000	¥ 240,000
Total direct costs						€ 7,858,000	¥ 31,432,000
Total realisation costs excl. VAT						€ 7,858,000	¥ 31,432,000
Gate type 3		63	Amount	€ 7,054,000	¥ 28,216,000	€ 444,402,000	¥ 1,777,608,000
Gate	1049 tons	1049	Metric ton	€ 6,000	¥ 24,000	€ 6,294,000	¥ 25,176,000
Barge		1	Amount	€ 50,000	¥ 200,000	€ 50,000	¥ 200,000
Crane		1	Amount	€ 50,000	¥ 200,000	€ 50,000	¥ 200,000
Supports and personnel		1	Amount	-	-	-	-
Hydraulics		1	Amount	€ 600,000	¥ 2,400,000	€ 600,000	¥ 2,400,000
Operation& control	10% of hydraulics	1	Amount	€ 60,000	¥ 240,000	€ 60,000	¥ 240,000
Total direct costs						€ 7,054,000	¥ 28,216,000
Total realisation costs excl. VAT						€ 7,054,000	¥ 28,216,000
Gate type 4		1	Amount	€ 1,770,000	¥ 7,080,000	€ 1,770,000	¥ 7,080,000
Gate	250 tons	250	Metric ton	€ 6,000	¥ 24,000	€ 1,500,000	¥ 6,000,000
Barge		1	Amount	€ 25,000	¥ 100,000	€ 25,000	¥ 100,000
Crane		1	Amount	€ 25,000	¥ 100,000	€ 25,000	¥ 100,000
Supports and personnel		1	Amount	-	-	-	-
Hydraulics		1	Amount	€ 200,000	¥ 800,000	€ 200,000	¥ 800,000
Operation& control	10% of hydraulics	1	Amount	€ 20,000	¥ 80,000	€ 20,000	¥ 80,000
Total direct costs						€ 1,770,000	¥ 7,080,000
Total realisation costs excl. VAT						€ 1,770,000	¥ 7,080,000
Total costs of gates excl. VAT						€ 776,208,000	¥ 3,104,832,000

11.2.3.2 Upstream bed protection

In this open option, the water depth at the bed protection, the river width and the thickness of the bed protection are equal to the gate option. Therefore, the estimated costs of upstream bed protection of open option are equal to the estimated costs of upstream bed protection of gate option.

Table 11- 12. Dimensions of upstream bed protection of 'open option'

Protection area	Parameter	Value	Unit
	Total rocks along bar	8,630,400	m3
	Total filter mats alon	1,087,500	m3
Protection area B1	Width	200	m
	Protection area	3,000,000	m2
	Rock 1-3 ton	Thickness 2.2	m
	Rock 60-300 kg	Thickness 0.86	m
	Rock 10-60 kg	Thickness 0.6	m
	Rock 90-250 mm	Thickness 0.3	m
	Filter mats	Thickness 0.5	m
Protection area B2	Width(m)	100	m
	Protection area (m2)	1,500,000	m2
	Rock 300-1000 kg	Thickness 1.1	m
	Rock 10-60 kg	Thickness 0.6	m
	Rock 90-250 mm	Thickness 0.3	m
	Filter mats	Thickness 0.25	m

Table 11- 13. Cost estimation of the upstream bed protection of 'open option'

Objects	Unit	Dimension	Unit	Dimension	Unit cost		Total costs	
Rock 1-3 ton	3,828,000	m3	10,144,200	tons	€ 30	¥ 200	€ 304,326,000	¥ 2,028,840,000
Rock 300-1000kg	957,000	m3	2,536,050	tons	€ 30	¥ 200	€ 76,081,500	¥ 507,210,000
Rock 60-300kg	1,496,400	m3	3,965,460	tons	€ 30	¥ 200	€ 118,963,800	¥ 793,092,000
Rock 10-60kg	1,566,000	m3	4,149,900	tons	€ 30	¥ 200	€ 124,497,000	¥ 829,980,000
Rock 90-250mm	783,000	m3	2,074,950	tons	€ 30	¥ 200	€ 62,248,500	¥ 414,990,000
Filter mats 0.25m thick	-	-	1,740,000	m2	€ 20	¥ 150	€ 34,800,000	¥ 261,000,000
Filter mats 0.5 m thick	-	-	870,000	m2	€ 20	¥ 150	€ 17,400,000	¥ 130,500,000
Total direct costs excl. VAT							€ 738,316,800	¥4,965,612,000

11.2.3.3 Downstream bed protection

High current velocities occur through the opening during floods. It is not cost-effective to construct the whole protection with the same element size, because the velocities decrease quickly behind the barrier. Further downstream of the flow, smaller elements can be applied, since the estuary is quite wide and the flow spreads and the intensity of the turbulence becomes much lower.

For economic reasons, it is decided to divide the protection area, which locates directly behind the opening, into three areas A1 (0-150m), A2 (150-350m) and A3 (350m-400m), more information is presented in Figure 10- 10. The computed downstream bed protection is presented in Table 11- 15.

Table 11- 14. Dimensions of downstream bed protection of 'open option'

Protection area	Parameter	Value	Unit
Protection area B1	Width	200	m
	Protection area	1,661,000	m2
	Rock 1-3 ton	Thickness 2.2	m
	Rock 60-300 kg	Thickness 0.86	m
	Rock 10-60 kg	Thickness 0.6	m
	Rock 90-250 mm	Thickness 0.3	m
	Filter mats	Thickness 0.5	m
Protection area B2	Width(m)	100	m
	Protection area	783,000	m2
	Rock 300-1000 kg	Thickness 1.1	m
	Rock 10-60 kg	Thickness 0.6	m
	Rock 90-250 mm	Thickness 0.3	m
	Filter mats	Thickness 0.25	m
Protection area A1	Width(m)	150	m
	Protection area	58,500	m2
	Rock 6-10 ton	Thickness 3.24	m
	Rock 60-300 kg	Thickness 0.86	m
	Rock 10-60 kg	Thickness 0.5	m
	Rock 45-200 mm	Thickness 0.5	m
	Filter mats	Thickness 0.5	m
Protection area A2	Width(m)	200	m
	Protection area	85,000	m2
	Rock 1-3 ton	Thickness 2.7	m
	Rock 10-60 kg	Thickness 0.3	m
	Rock 90-250 mm	Thickness 0.3	m
	Filter mats	Thickness 0.25	m
Protection area A3	Width(m)	50	m
	Protection area	22,500	m2
	Rock 300-1000 kg	Thickness 1.77	m
	Rock 10-60 kg	Thickness 0.6	m
	Rock 90-250 mm	Thickness 0.3	m
	Filter mats	Thickness 0.25	m

The unit price of each material is equal the upstream bed protection. The costs are estimated in Table 11- 15.

Table 11- 15. Cost estimation of downstream bed protection of 'open option'

Objects	Unit	Dimension	Unit	Dimension	Unit cost		Total costs	
Rock 6-10 ton	189,540	m3	502,281	tons	€ 30	¥ 200	€ 15,068,430	¥ 100,456,200
Rock 1-3 ton	3,883,700	m3	10,291,805	tons	€ 30	¥ 200	€ 308,754,150	¥ 2,058,361,000
Rock 300-1000kg	901,125	m3	2,387,981	tons	€ 30	¥ 200	€ 71,639,438	¥ 477,596,250
Rock 60-300kg	1,478,770	m3	3,918,741	tons	€ 30	¥ 200	€ 117,562,215	¥ 783,748,100
Rock 10-60kg	1,534,650	m3	4,066,823	tons	€ 30	¥ 200	€ 122,004,675	¥ 813,364,500
Rock 90-250mm	765,450	m3	2,028,443	tons	€ 30	¥ 200	€ 60,853,275	¥ 405,688,500
Rock 45-200 mm	29,250	m3	77,513	tons	€ 30	¥ 200	€ 2,325,375	¥ 15,502,500
Filter mats 0.25m thick	-	-	890,500	m2	€ 20	¥ 150	€ 17,810,000	¥ 133,575,000
Filter mats 0.5 m thick	-	-	1,719,500	m2	€ 20	¥ 150	€ 34,390,000	¥ 257,925,000
Total direct costs excl. VAT							€ 750,407,558	¥5,030,714,550

11.2.3.4 Protection through opening

The design of protection through opening is presented in Section 0. The stone-grouted box gabions are attached to the geotextile with plastic pins. The box gabions sink the mats and keep the mats stable. After sinking, the mats will be covered with steel slags for more stability in turbulent flows. The dimensions of mats are 50*375m^2.

Table 11- 16. Cost estimation of bed protection through opening of 'open option'

Objects	Specification	Unit	Dimension	Unit cost		Total costs	
Bed protection through opening	Area 375* 46.5 m2						
Box gabions	Size 1*1*2 m3	8718	Amount	€ 25	¥ 30	€ 217,950	¥ 261,540
	Rocks inside 60-300 kg	6320	tons	€ 30	¥ 200	€ 189,600	¥ 1,264,000
Geo-textile	Terrafix 420R Mats with O90=0.5mm k=0.05-0.5 (1/s) Roll size 4.57*91.44 m2	55	Amount	€ 20	¥ 150	€ 1,100	¥ 8,250
Total direct costs excl. VAT						€ 408,650	¥ 1,533,790

11.2.3.5 Foundation

The barrier has to be founded so as to transport loads to subsoil. Assuming the head and soil conditions are equal to the gate option, the pile plan of gate option can be applied. Steel tubular sheet piles are used, with diameter of 1 meter. Foundation piles will be driven to WD-65m into the bearable bed layer.

Table 11- 17. Cost estimation of foundation of 'open option'

Objects	Specification	Unit	Dimension	Unit cost		Total costs	
Foundation		1	Amount	€ 50,658,300	¥ 297,990,000	€ 50,658,300	¥ 297,990,000
Foundtion type 1 Tubular steel piles,d=1000mm Grid Pile length	 Slab 48*375 m2 4*4 m2 59.5m per pile	 18000 540 32130	 m2 Amount m	€ 85	¥ 500	€ 2,731,050	¥ 16,065,000
Foundtion type 2 Tubular steel piles,d=1000mm Grid Pile length	 Slab 42*(20*80) m2 4*4 m2 52.5m per pile	 67200 4200 220500	 m2 Amount m	€ 85	¥ 500	€ 18,742,500	¥ 110,250,000
Foundtion type 3 Tubular steel piles,d=1000mm Grid Pile length	 Slab 63*(20*80) m2 4*4 m2 54.5m per pile	 100800 6300 343350	 m2 Amount m	€ 85	¥ 500	€ 29,184,750	¥ 171,675,000
Total direct costs excl. VAT						€ 50,658,300	¥ 297,990,000

11.2.3.6 Civil works

The estimated costs of supporting structures are related to the amount of concrete used. The determination of costs of connecting levees is based on levee dimensions. An estimate of such civil works is presented in Table 11- 18.

Table 11- 18. Cost estimation of civil works of 'open option'

Objects	Specification	Unit Dimension	Unit cost		Total costs	
Supporting structures						
Base slab		252631 m3				
Total volumn of concrete		252631 m3	€ 680	¥ 2,720	€ 171,789,080	¥ 687,156,320
Levees						
Levee along banks		20 km				
Connecting levee		68 km				
Total volumn of concrete		78.32 km	€ 5,000,000	¥ 20,000,000	€ 391,600,000	¥ 1,566,400,000
Total direct costs excl. VAT					€ 563,389,080	¥ 2,253,556,320

11.2.3.7 Construction pit

A construction pit has to be created for the barrier realization. It is decided to have artificial islands, one at the navigational section, so the island can also serve as approaching breakwaters for vessels. The island should have the minimum area of 200*50 m^2. A second island is decided to be in the middle of barrier of the North Channel. The unit cost is roughly estimated as 500,000 euros per island.

Table 11- 19. Cost estimation of construction pit of 'open option'

Objects	Specification	Unit Dimension	Unit cost		Total costs	
Construction pit		2 Amount	€ 500,000	¥ 3,500,000	€ 1,000,000	¥ 14,000,000
Total direct cost			€ 500,000	¥ 3,500,000	€ 1,000,000	¥ 14,000,000
Total direct costs excl. VAT					€ 1,000,000	¥ 14,000,000

11.3 Comparison of cost estimations

Table 11- 20 presents the estimated costs of the barrier system in the Yangtze Estuary. The substantiation of the cost of each option is described in previous sections. Costs are estimated for a pre-feasibility design level. The estimated costs are a first indication of the real realization costs. As many uncertainties may occur during the execution of the project, it is impossible to estimate the costs accurately. To increase the accuracy, more information is required, especially the local conditions around the barrier location and surroundings.

Table 11- 20. Cost estimations of both options

Gate option			Open option		
Objects	Costs (million)		Objects	Costs (million)	
Gates	€ 811	¥ 3,245	Gates	€ 776	¥ 3,105
Upstream bed protection	€ 738	¥ 4,966	Upstream bed protection	€ 738	¥ 4,966
Downstream bed protection	€ 738	¥ 4,966	Downstream bed protection	€ 750	¥ 5,031
-	-	-	Bed protection through opening	€ 0	¥ 2
Seepage cut off	€ 101	¥ 168	Seepage cut off	€ 99	¥ 165
Foundation	€ 52	¥ 303	Foundation	€ 51	¥ 298
Civil works	€ 515	¥ 2,059	Civil works	€ 563	¥ 2,254
Construction pit	€ 1	¥ 14	Construction pit	€ 1	¥ 14
Total direct costs of barrier system excl. VAT:			**Total direct costs of barrier system excl. VAT:**		
	€ 2,956	**¥ 15,721**		**€ 2,979**	**¥ 15,833**
				0.79%	**0.71%**

Remarks:

- *Price with [red box] means 'bad', the option is more expensive*
  Price with [green box] means 'good', the option is cheaper
  Price with [yellow box] means 'neutral', both options cost the same

- *The barrier system should be considered as part of the city's flood protection strategy. The banks around the barrier location are very densely populated with several buildings and small ports. Considering this characteristic, implementing the defense strategy appears to be very complicated, which is expected to result in higher cost. These costs are not taken into account.*
- *The top levels of different sections differ. In order to obtain a storm surge defense system, the banks and quays have to be raised. Also a water retaining structure should be considered as connection of different sections. These would result in major construction works, like raising roads, constructing levees etc. These costs are not included in this report.*
- *Costs of construction methods high depend on local available equipment and the seasonal variety. These costs are not taken into consideration.*
- *Additional costs like research, permits, migration etc. are not included in this report.*

It is common to construct a movable gate in the navigational section, which result in the 'gate option'. If the water level inside the estuary is kept under its maximum water level, the open option can also be applied. Obviously, without the movable gate, the costs of gates are reduced significantly, from 811 million euros to 776 million euros. But the high current velocities through and behind the opening require heavy bed protection. It can be seen from Table 11- 20, the downstream bed protection raises the total costs a lot. In addition, the costs of other objects also vary due to different design philosophy. Considering all these direct costs, the open option is even around 1% more expensive than the gate option.

11.4 Conclusion

The overall design of both the gate option and the open option are reviewed in this chapter. Then a cost estimation is performed mainly focus on the main parts of the barrier systems.

The results of the estimations shown, even the open option consist less gates than the gate option, the total cost of the open option is still around 1% higher. That is because much heavier bed protection work is required to sustain large current velocities behind the opening. This caused the total costs of the open option to increase a lot, which already encounters the reduced costs on gates. But the 1% difference can be actually neglected, because the accuracy level of the estimations is quite low. Which one is the better choice still requires further research.

12. CONCLSIONS & RECOMMENDATIONS

12.1. Conclusion

In this report the conceptual design of the flood protection system of Shanghai has been considered. The main research question was as follows:

"What is the most suitable, reliable and economical option to protect Shanghai from flooding, and the conceptual design for the navigational section?"

Following conclusions are derived from this thesis report in response to this research question.

<u>Design level 1- Flood defense system to protect Shanghai from flooding</u>

Considering the program of requirements of boundary conditions, several solutions have considered and evaluated for the protection system. Between all the alternatives, by using MCA and rough cost indication, the barrier system closing off the Yangtze Estuary has been selected as the best choice, which fulfills all the requirements of the project.

This barrier system consists of three storm surge barriers behind the islands in the Yangtze Estuary. It is suitable for the complicated system, providing large protected area, has reasonable construction and maintenance costs and guarantees a long-term safety to the whole region.

<u>Design level 2- Barrier system in the Yangtze Estuary</u>

Design level 2 discussed the operational and functional requirements for the selected barrier system. Except its fundamental function a flood defense structure, two main basic functions must be satisfied. One is to allow free shipping under all conditions; the other one is to exchange water between the estuary and the sea so as to preserve the ecosystem. Then the barrier system is divided into two main functional sections: navigational section and environmental section.

The navigable requirements of all three channels in Yangtze Estuary have been calculated. The required environmental section is obtained by using the 'storage basin approximation'. That results in a maximum closure of 40% of the original area during normal conditions. After that, several options have been proposed and assessed for the distribution of the two functional sections between the three channels. The selection is made on the basis of economic considerations. It has been decided not to have navigational section in the North Channel, but only one exists in the South Channel to save money.

<u>Design level 3- Barrier in the South Channel</u>

The part of barrier in the South Channel consists of both environmental and navigational sections. First, a general design for the environmental section is performed, by using MCA. The vertically lifting gates are selected, because they are feasible for large span and suited to reverse differential head and reverse flow during operation. The main global structural dimensions are determined.

The navigational section can be either open-closable or permanently open. With a closable gate in the navigational section, the minimum opening width for the free navigation is 172m. Otherwise, for the open option, the maximum opening width is 375m to ensure no floods occur behind the barrier.

Design level 4- Open or closable navigational section in the South Channel

Design level 4 focuses on the navigational section in the South Channel. For the gate option, the barge gate is selected. The barge gate is suitable for the wide openings, provide unlimited air draft and in the floating situation doesn't transfer too much loads to the foundations. The barge gate is designed in more detail, including the materials, supporting structures, operation system, foundation and bed protection etc.

The main difference between the gate option and open option lies in the bed protection behind the navigational section. With a closed barge gate, the current field is softer than open option during storm conditions. Heavy bed protection with large rocks and several filter layers is proposed for the open option, with the protection length of 400 meters.

Then a cost estimation is performed mainly focus on the main parts of the barrier systems.

The estimation is performed focusing on the main parts of the barrier systems. The results reveal the total costs of open option are around 1% higher. That is because much heavier bed protection work is required to sustain large current velocities behind the opening. This caused the total costs of the open option to increase a lot, which even encounters the reduced costs due to less required gates.

12.2. Recommendations

- This report is based on design storm with the design return period of 1/1,000 [1/year]. But the design storm and required safety level should be considered in more detail, using the probabilistic design approaches. It could be useful to take the cost-benefit analysis.
- The Yangtze Estuary is a quite complicated system. The surrounding water system, including the Yangtze River, the Huangpu River and the Tai Lake would also influence the design philosophy. So laboratory models and tests should be realized to study more about the barrier system and operations in reality.
- A modified discrete system is used for determination on required flow area. However, whether or not and how the river discharge would influence the theory is not yet

known. Further research is recommended to be done with the help with more advanced software such as Delft 3D or Sobek making hydrological models.

- When designing the barge gate for the gate option, only the hydrostatic loads are considered. Extensive studies of hydrodynamic loads are recommended.
- The barrier system should be considered as part of the city's flood protection strategy. The banks around the barrier location are very densely populated with several buildings and small ports. Implementing the defense strategy should be considered.
- The top levels of different sections of the barrier system differ. In order to obtain a storm surge defense system, the banks and quays have to be raised. Also a water retaining structure should be considered as connection of different sections.
- The comparison between the gate option and open option mainly focuses on the costs in this report. Further study including more factors must be considered. MCA carried out by virtual stakeholders can be useful.

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14. APPENDIX

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Appendix A. Shanghai Flood Defense System

According to the National Standard *"Standard for Flood Control, GB50201-94. (1995)"*, the primary flood retaining structures around Shanghai have to provide full protection against floods with a return period of 1,000 years. To protect Shanghai against flood disasters, the local government has reinforced and extended its defense lines as described in following paragraphs.

A.1. Dikes

The first lifeline to protect Shanghai from typhoon is the front sea dike along the shoreline of the continent in Shanghai. It is 542.8 km long, including the 464.4 km long national dike. Take the dikes in Fengxian District as an example; this part locates in the middle of the Hangzhou Bay (see Figure A- 1). The seaside of the dike is made of riprap, with an outer slope of 1:3. The dike crest is at the elevation of 9.5 m to 10 m WD and 10 m wide (Zhang, 2009).

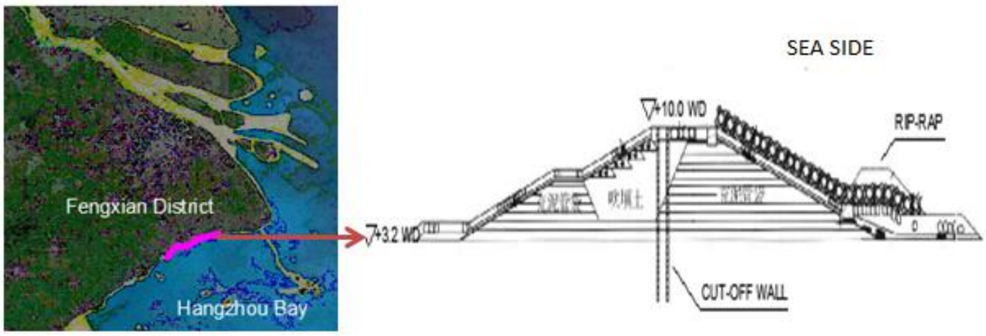


Figure A- 1. Typical Cross Section of Sea Dike in Fengxian District

According to the present tidal prevention criteria (a return period of 200 years for a high tidal level plus 11-level wind), only two thirds of the dike meets the requirement. The sea walls to protect coastal industries (like the Baoshan steel plant), ports and areas for travelling, are of first-level dikes. They should adopt the defense criteria of once in a thousand years.

A.2. Flood Wall

Before the year 1949[1], there was no special flood prevention engineering facilities, except at some parts of the Huangpu River and the Suzhou River were there any bank protection works with a height of 4.7 m. In July 1949, the water level rose to 4.77 m, due to a typhoon, then the total area was submerged. After that, the local industry developed rapidly and the factories extracted large amount of ground water which led to further subsiding of the city and the water level during the flood period often increased to the dikes' top(Cheng, 2004).

After 1956, simple flood prevention walls and soil dikes had been constructed which were 50 km long and 4.8 m high, but they were destroyed by a storm in 1962. Then local people began

[1] The Chinese Communist Party gained power and the Central Government of the People's Republic of China was established in 1949.

to build the floodwalls to a much greater extent. In the following decades, the local government continuously constructed and heightened the dikes in order to meet the increasing demands for flood protection. The history of heightening the floodwall is show in Table A- 1.

Table A- 1. History of Heightening Flood Walls in Shanghai (Source: (ESCAP))

Year	Height(m)	Length(km)
1949	4.7	Quite small part
1956	4.8	50
1974	5.2	120
1981	5.8	186
1994	6.9	208

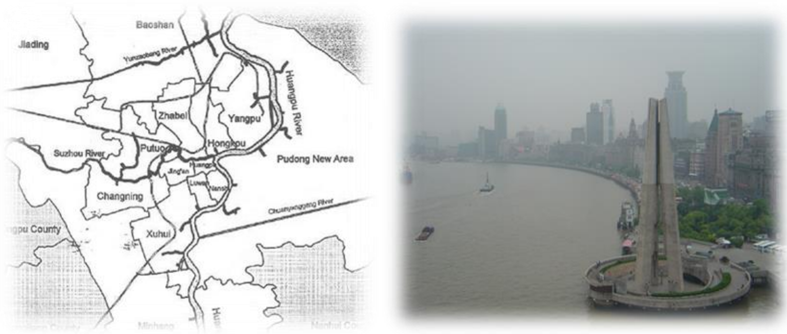


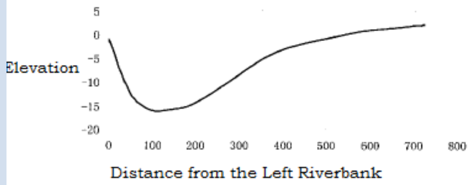
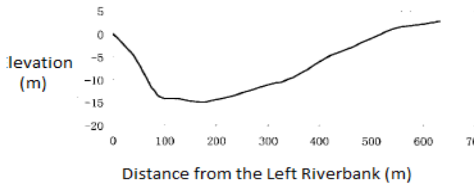
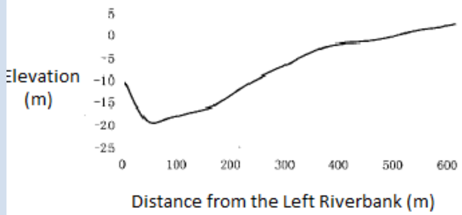
Figure A- 2. Urban Flood Wall in Shanghai

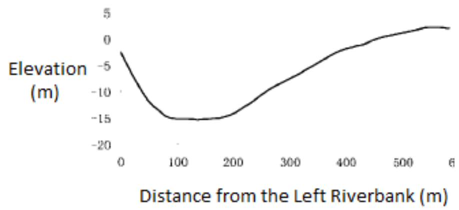
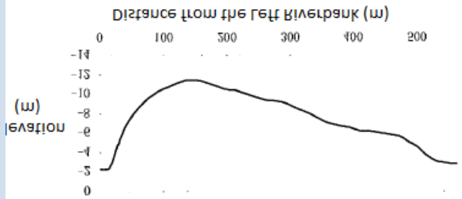
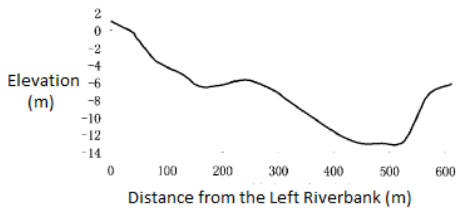
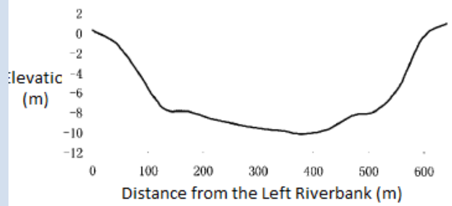
A.3. Drainage

Sea dikes and floodwalls can only withstand the external floods, but the inner city waterlogging is another sever problem in urban areas in Shanghai. Liu (2006) studied the relationship between poor drainage systems with urban flooding probability in Shanghai. As the relief in Shanghai is very low, in addition, the land is subsiding at the same time, so it is easy to form waterlogged areas. The Shanghai Municipal Government has already built hundreds of pumping stations to help strengthen the drainage capacity(Liu, Guo, Zhang, & Lv, 2006). However, due to the low drainage criteria, there still exists a risk when the rainstorms and tides come together.

Appendix B. Proposed barrier by Shanghai Municipal Government

According to the project planning research by the Shanghai Water Authority, 7 locations are put forward in the mouth. This part gives more information about the proposed barrier location, which corresponds to Section 4.2 in the main report.

Location	Width (m)	Distance to the mouth (km)	Description	Cross Section Configuration
Wusongkou	750	0.00	The first location is just at the crossing point of the Huangpu River and the Yangtze River. The advantage of this location is that it protects all the tributaries of the Huangpu River and makes the river the largest storage capacity. However, there is a naval base just located behind this point, which makes it a prohibitive factor.	
Wukong Park	650	1.40	This location is situated just downstream of Wencao Bang, that it reduces the flood probability to the city center. A central ferry terminal is constructed in this area. A 500V supervltage transmission power line has just been wired in 2006.	
Zhanghua Bang	550	2.80	This location is at the river bend, which is unfavorable for shipping. This site is densely populated with industries, besides the Shanghai Container Terminals is already there.	

Jungong Road	550	4.00	This location is between a large ship company and a small port. What' s more, as planned by the local government, the Pudong Railway and Yangtze West Tunnel will be constructed through this site.	
Changhang	550	5.08	This location is just within the ship turning basin. There are also many industries along the bank.	
Fishery Yard	600	8.75	This location is a straight section, although it is just in front of a bend. As its name shows, a fishery yard is close to it. There is no large port along.	
Forest Park	625	12.00	This is the most upstream barrier location, which is 12 km from the river mouth. This is the most unfavorable regarding to the storage capacity confined by the barrier.	

Appendix C. Barrier gate type

C.1 Navigable gate types

In the second half of the 20st century storm surge barriers were constructed. In coastal cities, these barriers are installed to enhance coastal protection for urbanized areas at risk of flooding, under the impact of climate change and sea level rise. Paulo (2004) has covered very comprehensively in his book "*Design of Hydraulic Gates*", including the history, conception, construction and operation of almost all kinds of hydraulic gates. Since this study is aimed to design a suitable storm surge barrier for Shanghai, where navigation is of great importance, only navigable storm surge barriers that allow free shipping, are picked out from the list. Gates presented are compared with different aspects, including gate structures, environment aspects, navigation aspects, economical aspects, operation and maintenance.

In this part, six gate types that allow free navigation under normal conditions are explained with typical examples of nice performance:

- Mitre gates (double-leaf gate)
- Vertical lifting gates
- Flap gates
- Horizontal rotating or moving gates
- Vertical rotating gates
- Inflatable rubber dam

Mitre gates (double-leaf gate)

Mitre gates, the most classic type, were very common during the 17th century and are often used for ship locks. Mitre gates are double-leaf gates that leaves pointing upstream when the gates are close(Dircke, Jongeling, & Jansen, 2013). The schematic drawing with basic functional principles is shown in the following figure.

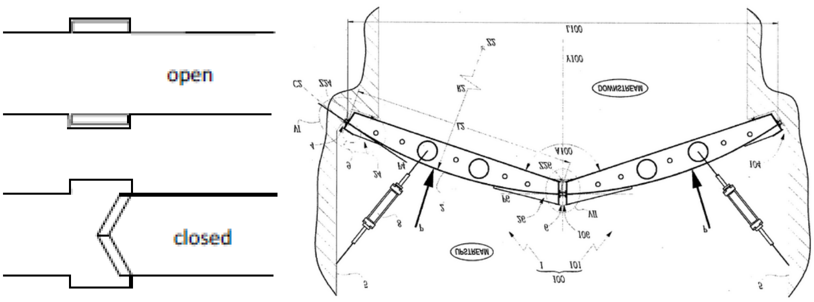


Figure C- 1. Schematic Drawing of Mitre Gate with Basic Functional Principles

Typically, the combined lengths of leaves exceed the lock width by 10 percent. A well-known example is the Ijmuiden complex in the Netherlands, connecting Amsterdam with the North

Sea (see Figure C- 1). This complex has four locks. The biggest lock (50 m) has rolling gates, while the other three (20 m) have mitre gates.

Figure C- 2. Ijmuiden Complex and Its Mitre Gate

Vertical lifting gates

The vertical lifting gate is the simplest type of flat gate. There is always a gate leaf that slides along the side guides fastened to the concrete. Thanks to its simple and safe operation, vertical lifting gates are widely used and much experience of design and construction is available. The concrete part, including the hoisting towers and its foundation are usually installed with cofferdams.

Because of its convenient installation and safe operation, the vertical lifting gate is often part of a large complex. A typical example is the new Seabrook Floodgate Complex, which are design to retain the water from the Lake Pontchartrain. There are three gates making the whole complex, one large sector gate and two small vertical lifting gate (see Figure C- 3).

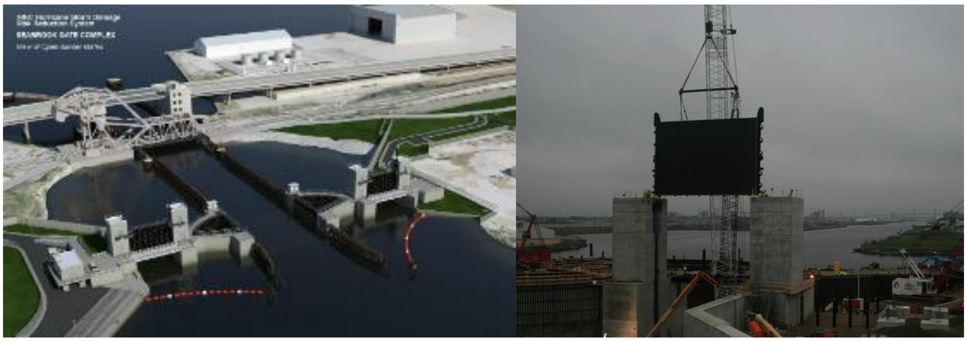


Figure C- 3. The New Seabrook Floodgate Complex and Its Lifting Gate, the USA

Flap gates

Flap gates are invisible when the barrier is not in use. The gate is stored in a bottom access with one end hinged on the sill. The other free end emerges above water when the barrier is closed. Due to the difficulty in inspection and maintenance, the flap gates are not that widely used. In addition, the silting of recess is another problem.

An example of flap gates is the Mose Buoyant Flap Gate in Venice. The complete Mose barrier consists of 78 hollow gates connecting the three inlets to the lagoon. At rest, the gate fills with water and lies flat under water on the sills. In the event of high water levels, compressed air forces water out, raising the gates (see Figure C- 4). Another example is the Eastern Scheldt Storm Surge Barrier, the largest project of the Delta Plan in the Netherlands.

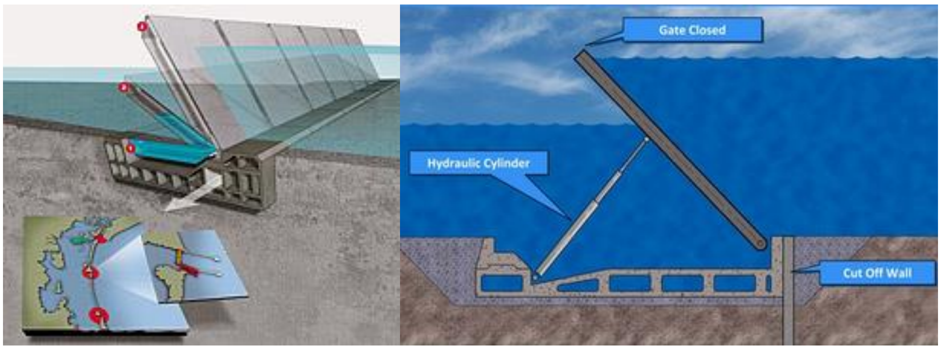


Figure C- 4. Mose Buoyant Flap Gate, Venice

Horizontal rotating or moving gate (including swing gate and sector gate, non-floating)

Swing gates are stored on one side of a waterway and pivots about a vertical axial to close. The flood control barrier in Louisiana, USA is shown in following figure. The Bayou Dularge Barge Gate spans 18.3 m and 6.25 m high. There is also an innovative concept of floating rotating barrier in Belgium and the Netherlands, with large spans up to 400.

Figure C- 5. Swing Barriers. Left: Bayou Dularge; Right: Bayou Lafourche, USA

Vertical axis sector gates are circular section. The sector gate has a curved skin plate, and its upper portion is a full surface in the radial direction, which makes it different from the segment gate. The advantage of this kind of gate is that, there is very litter unbalanced load and thus they can be closed and opened with differential head across the gate. One typical example is the Maeslant storm surge barrier, with 360 m length.

Figure C- 6. Maeslant Storm Surge Barrier, the Netherlands. Left: Closed; Right: Opened.

Vertical rotating gate (including segment gate and radial gate)

The segment gate consists of a curved skin plate, supported by radial steel arms. The segment gate rotate around a horizontal axis, which hinged to the bearing center and passing through the skin plate center. The kind of system makes the resultant trust go through the rotation point and it makes the open and closure process free of water pressure. An example is the Thames Barrier, which is located downstream of central London, preventing London from being flooded by extremely high tides and storm surges. How the barrier works is shown in Figure C- 7.

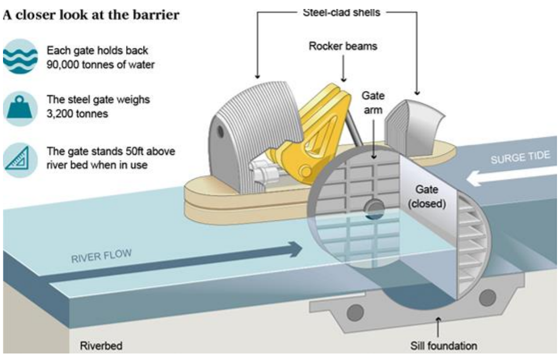


Figure C- 7. The Thames Barrier Operation System, London

A radial gate has a skin plate mounted on an open steel frame. For example, the Federal State Schleswig-Holstein in Germany have two radial gates, which are provided on each side of two lock chamber to supply flood protection with free navigation. Another example is that, there are lots of locks and dams in Belgium, consisting of radial gates to improve navigation.

Figure C- 8. Left: Federa State Schleswig- Holstein barrier, Germany; Right: Upper Meuse Basin, Belgium

Inflatable rubber dam

The inflatable rubber dams consist of long bladders, fixed to a bottom foundation. The dams are raised by inflating with water and/or air. Rubber dams are widely used around the world, but mainly for water control applications. It has not been constructed in deep water, possibly because of its sensitive materials to wave and water induced vibration. In addition, the fabrication of reinforced rubber sheet is difficult, especially with such large dimensions. The Romspol Storm Surge Barrier in the Netherlands, to prevent storm surges from the IJssel Lake, is the only major flood protection barrier consisting of inflatable rubber dams. Also, it is the largest one in Europe.

Figure C- 9. Romspol Storm Surge Barier, the Netherlands

Figure C- 10. Huaihua Dam, Qingdao City, China

C.2 Other gate types

This section discusses the barrier gate types without unlimited height clearance.

Visor gate

The visor gate is designed with a leaf with a three-hinged arc. In its closed position, the leaf presses continuously against the sill. In the open position, it limits the allowable air draft. The closure is done by gravity, while mechanical hoists make the opening.

Figure C- 11. Front view of a Visor Gate in Osaka, Japan (Left: open; Right: closed)

Caisson structure

The caisson structure is a concrete closable closure dam. It is permeable in normal conditions and closed during storm surges.

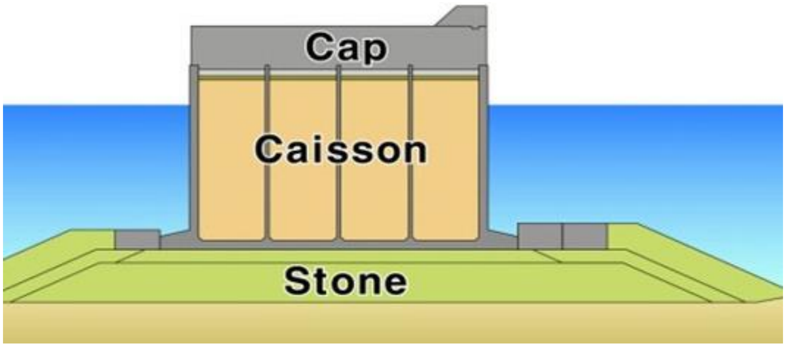


Figure C- 12. *Schematic birds eye view caisson structures*

Appendix D. Typhoons in Shanghai

Typhoons coming from the western North Pacific (WNP) mostly hit Shanghai during the boreal summer (June to September) every year. This name comes from a Chinese word Tai-fung meaning a great wind from the sea. They always come accompanied by heavy rains. The Japan's National Institute of Informatics (NII) gives the definition of Typhoon as a tropical cyclone (hereafter referred as TC) with the maximum wind of 34 knots[2] or higher. Therefore, it is quite necessary to understand the characteristics of tropical cyclones influencing Shanghai and associated storm surges will be analyzed in this section.

Formation of Tropical Cyclones

The main requirements to form a typhoon are pre-existing atmospheric disturbance, sufficiently warm sea surface temperatures, and high humidity, enough Coriolis force for a low pressure center, low vertical wind shear and upper atmosphere divergence. Normally the surface water should be at least 26 ^{o}C (79.9 ^{o} F) to maintain the warm core that fuels the tropical system(DeMaria, 1996).

Classification of Tropical Cyclones

According to the World Meteorological Organization (WMO), the intensity of a TC is classified by the maximum sustained wind. There are different standards for the classification of TCs. In this study, both the International standard (the Saffir-Simpson Hurricane Wind Scale) and Chinese standard are explained as follows.

The Saffir-Simpson Scale is shown in Table F- 1. Among the categories, Tropical Depression (TD) is a TC weaker than typhoon, which means that maximum sustained surface wind is less than 10 minutes. When the maximum sustained wind speed reached at least 10 minutes on average, a Tropical Storm (TS) forms. When the wind speed subsequently reaches 33m/s, a typhoon occurs.

Table F- 1. Classification of Tropical Cyclones according to the Saffir-Simpson Hurricane Wind Scale

Category	Max. Sustained Wind Speed	
	m/s	knots
TD	0-17	0-34
TS	18-32	35-63
1	33-42	64-82
2	43-49	83-95
3	50-58	96-113
4	59-69	114-135
5	>70	>136

[2] Knot is a unit for speed. One knot means a speed of moving one nautical mile (nm) in one hour.

In mainland China, a different method of classification is set up by the Chinese Academy of Meteorological Sciences (GBT 19201-2006), take an example, a typhoon with maximum winds stronger than 51.0 m/s is called Super typhoon (see Table F- 2).

Table F- 2. Chinese Standard of Tropical Cyclones Classification

Category	Max. Sustained Wind Speed (m/s)
Super Typhoon (Super TY)	<51.0
Strong Typhoon (STY)	41.5-50.9
Typhoon (TY)	32.7-41.4
Severe Tropical Storm (STS)	24.5-32.6
Tropical Storm (TS)	17.2-24.4
Tropical Depression (TD)	10.8-17.1

Whatever the methods of classification, almost any typhoon approach the Huangpu River can cause waterlogging in Shanghai urban area. Studying from the historical events, if the tropical cyclone reached the typhoon strength, it could cause flooding in the Huangpu River.

Structures of Tropical Cyclones

In generally, the size of typhoon is very large, with a radius of 200 to 300 kilometers. The circle-like isobaric lines are often used to describe the location and size of a typhoon on weather charts(Lander, 1994). Typhoon Haikui attacked east China in 2012.,Iit can be seen the typhoon is roughly a round, spiral-shaped whirling cloud. As it happened in the Northern Hemisphere, the wind flew in counterclockwise direction surrounding its center. Eye of typhoon, the center of its huge cloud column, is cloudless, windless and rainless. However, the area just outside the eye, has the thickest clouds and the most intense winds, plus heaviest rains. That area is called the eye wall.

Decay of Tropical Cyclones

Tropical cyclones evolve through a various life span, from the formation to dissipation. The life span could only be one or two days as the shortest. It can even reach two weeks; and average around five days. It is determined by the supply of moisture and heat, when it comes into cooler seawaters. Besides, the wind shear induced surface friction can reduce the circulation as well.

Appendix E. Cost index number definition

According to Ad van der Toorn (2010), the cost of a new barrier or other hydraulic structures can be roughly estimated with the so called index number/ unity price. The index number for calculating barrier, dike and floodwall are illustrated below separately.

Barrier

The cost of a new barrier is strongly related with the width, the retaining height and the head over the structure. So in this case, the index number has the dimension [€/m^3]. Based on that consideration, Ad van der Toorn estimated a group of barriers inside and outside of the Netherlands, the index number is more or less the mean value for different barriers(van der Toorn, 2010). The barrier factor (2010: 30,000 €/m^3) is modified by a 2% inflation rate over the three years, resulting in 31,836 €/m^3. Considering the real situation in China, the materials and labor cost are much lower, while the construction is more complex, 50 % of the price in the European countries is assumed. Thus the index number for a new barrier is defined as 15,918 €/m^3. However, the actual cost can be different according to the gate opening (the index number will be lower if there are a lot of nearly the same gates) or some special reasons (e.g. the barrier in st.-Petersburg has a special opening designed for navy).

5

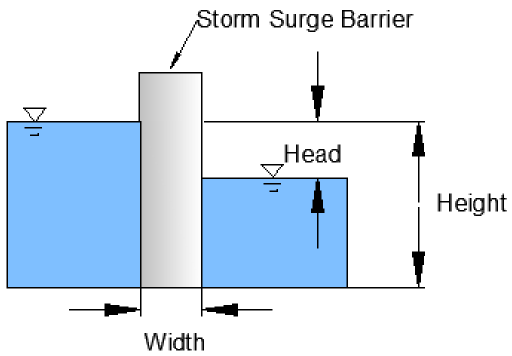


Figure E- 1. Simplified Storm Surge Barrier Cross Section

Dike rising

The average height of existing dikes is around 3.5 to 4.5 m. To resist the expected floods in 50 years, the dikes should be raised by 1.75m on average(Yongxing, Shuiqin, & Aoquan, 2002). The cost of dike rising can be expressed in cost per kilometer in length. Ad van der Toorn has estimated a rather homogeneous group of released dike raising projects. In the real world, the difference between simple dike raising project and dike reinforcement in the water and wave zone should be paid attention. In addition, the construction of inner side of the dike can be more expensive if there are local building situated beside the dike, making the construction more complex. But this part stays unclear, as more detailed information about the existing dikes in Shanghai is not available. By extrapolating the index number line to 1.75 m, the cost of dike rising in the Netherlands is 9 million Euros per meter per kilometer dike in 2010 (see Figure E- 2). Modifying by a 2% inflation rate over the three years, 9.55 million Euros per meter per kilometer is obtained. Regarding the price level difference in China, the index number (unity cost) for dike rising in this study is adopted as half of the Dutch price, as 5,000,000 €/km.

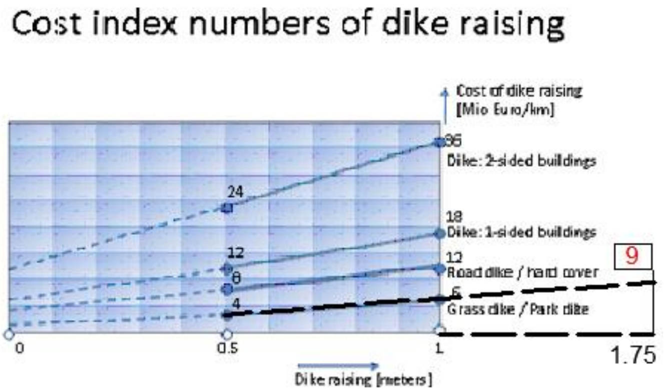


Figure E- 2. Cost Index Number of Dike Raising (Source: (van der Toorn, 2010))

Floodwall

The floodwall along the Huangpu River should be reconstructed other than heightening due to its poor quality and management in the last few decades. According to W.F. Molenaar (Personal Conversation, March 2014), a simple retaining floodwall cross section can be used for rough estimation, see Figure E- 3. The height of floodwall above the ground level is the same as that under the soil. The width is assumed to be 1 meter. The construction price of such a floodwall in Europe, the Netherlands in particular, is around 350 €/m3. For this floodwall, the unity price is 3,500 €/m. Considering there are price level difference between the Netherlands and China, 1,750 €/m is used in this study. The floodwall has to be built through the economical center of Shanghai, where the actual construction would be greatly more difficult than in the rural area. Concerning that aspect, the final index number is defined as 15,000 €/m.

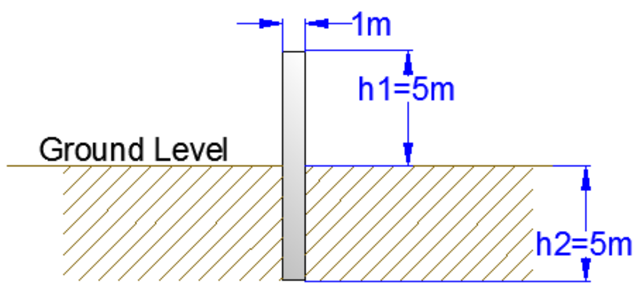


Figure E- 3. Typical Floodwall Cross Section Applied to Shanghai

Appendix F. Multi criteria analysis (barrier system selection)

F.1 Assignment of weighing factors

The weighing assignments are carried out by the main five stakeholders as stated before. The separate results are shown in this part.

Table F- 3. Weighing Result from Shanghai Municipal Government

Criteria		Flood Safety		Construction			Ecological impacts			Economic impacts			Relative weights
		1	2	3	4	5	6	7	8	9	10	11	
Flood Safety	A		1	1	1	1	1	1	1	1	1	1	18%
	B	0		1	1	1	1	1	1	0	0	0	11%
Construction	C	0	0		0	0	1	0	0	0	0	0	2%
	D	0	0	1		0	1	1	0	0	0	0	5%
	E	0	0	1	1		1	1	0	0	0	1	9%
Ecological impacts	F	0	0	0	0	0		0	1	0	0	1	4%
	G	0	0	1	0	0	1		0	0	0	1	5%
	H	0	0	1	1	1	0	1		1	0	1	11%
Economic impacts	I	0	1	1	1	1	1	1	0		1	1	15%
	J	0	1	1	1	1	1	1	1	0		1	15%
	K	0	1	1	1	0	0	0	0	0	0		5%
													100%

Table F- 4. Weighing Result from Ports or Shipping-related Companies

Criteria		Flood Safety		Construction			Ecological impacts			Economic impacts			Relative weights
		1	2	3	4	5	6	7	8	9	10	11	
Flood Safety	A		1	1	1	1	1	1	1	1	1	1	18%
	B	0		1	1	1	1	1	1	0	0	0	11%
Construction	C	0	0		0	0	1	0	0	0	0	0	2%
	D	0	0	1		0	1	1	0	0	0	0	5%
	E	0	0	1	1		1	1	0	0	0	1	9%
Ecological impacts	F	0	0	0	0	0		0	1	0	0	1	4%
	G	0	0	1	0	0	1		0	0	0	1	5%
	H	0	0	1	1	1	0	1		1	0	1	11%
Economic impacts	I	0	1	1	1	1	1	1	0		1	1	15%
	J	0	1	1	1	1	1	1	1	0		1	15%
	K	0	1	1	1	0	0	0	0	0	0		5%
													100%

Table F- 5. Weighing Result from Residents Living/ Working in Shanghai Urban Area

Criteria		Flood Safety		Construction			Ecological impacts			Economic impacts			Relative weights
		1	2	3	4	5	6	7	8	9	10	11	
Flood Safety	A		1	1	1	1	1	1	1	1	1	1	18%
	B	0		1	1	0	0	0	0	1	0	0	5%
Construction	C	0	0		0	0	0	0	1	0	0	0	2%
	D	0	0	1		0	0	1	1	0	0	0	5%
	E	0	1	1	1		1	0	1	0	1	1	13%
Ecological impacts	F	0	1	1	1	0		1	1	1	1	1	15%
	G	0	1	1	0	1	0		1	0	1	0	9%
	H	0	1	0	0	0	0	0		1	0	0	4%
Economic impacts	I	0	0	1	1	1	0	1	0		1	0	9%
	J	0	1	1	1	0	0	0	1	0		0	7%
	K	0	1	1	1	0	0	1	1	1	1		13%
													100%

Table F- 6. Weighing Result from Residents Living/ Working on Chongming Island

Criteria		Flood Safety		Construction			Ecological impacts			Economic impacts			Relative weights
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	
Flood Safety	**A**		1	1	1	1	1	1	1	1	1	1	18%
	B	0		1	1	0	0	0	0	1	0	0	5%
Construction	**C**	0	0		0	0	0	0	1	0	0	0	2%
	D	0	0	1		0	0	1	1	0	0	0	5%
	E	0	1	1	1		1	0	1	0	1	1	13%
Ecological impacts	**F**	0	1	1	1	0		1	1	0	1	1	13%
	G	0	1	1	0	1	0		1	0	1	1	11%
	H	0	1	0	0	0	0	0		1	1	0	5%
Economic impacts	**I**	0	0	1	1	1	1	1	0		1	1	13%
	J	0	1	1	1	0	0	0	0	0		0	5%
	K	0	1	1	1	0	0	1	0	0	1		9%
													100%

Table F- 7. Weighing Result from Environmental Parties

Criteria		Flood Safety		Construction			Ecological impacts			Economic impacts			Relative weights
		1	2	3	4	5	6	7	8	9	10	11	
Flood Safety	A		1	0	1	1	1	1	1	1	1	1	16%
	B	0		0	1	1	0	0	0	0	0	0	4%
Construction	C	1	1		1	0	0	0	0	0	0	0	5%
	D	0	0	0		0	0	0	0	0	0	0	0%
	E	0	0	1	1		0	1	0	0	1	1	9%
Ecological impacts	F	0	1	1	1	1		1	1	1	1	1	16%
	G	0	1	1	1	0	0		1	1	1	1	13%
	H	0	1	1	1	1	0	0		1	0	0	9%
Economic impacts	I	0	1	1	1	1	0	0	0		0	1	9%
	J	0	1	1	1	0	0	0	1	1		0	9%
	K	0	1	1	1	0	0	0	1	0	1		9%
													100%

F.2 Scoring results of proposed options for flood defense system

Two Chinese hydraulic engineers, and a citizen from Shanghai urban area finish the scoring assignment. The separate scoring results are shown in this part.

Table F- 8. Scoring Result from Profession I

Criteria			Relative Weights(%)	Scores				Weighed Scores			
				1	2	3	4	1	2	3	4
Flood Safety	A	System failure	17.0	1.0	1.0	2.0	3.0	17.0	17.0	34.0	51.0
	B	Flexibility	5.5	1.0	1.0	2.0	2.0	5.5	5.5	10.9	10.9
Construction	C	Materials	2.5	2.0	2.0	2.0	2.0	5.1	5.1	5.1	5.1
	D	Accessibility	5.0	4.0	4.0	3.0	2.0	19.9	19.9	14.9	10.0
	E	Impact on populated area	11.0	3.0	3.0	3.0	3.0	32.9	32.9	32.9	32.9
Ecological impacts	F	Pollution on populated area	11.3	2.0	3.0	2.0	2.0	22.5	33.8	22.5	22.5
	G	Preservation of ecosystem	10.3	0.0	0.0	3.0	2.0	0.0	0.0	30.8	20.5
	H	Salt water intrusion	6.9	4.0	4.0	2.0	2.0	27.5	27.5	13.7	13.7
Economic impacts	I	Protected area&population	10.9	2.0	2.0	2.0	2.0	21.7	21.7	21.7	21.7
	J	Disturbance of navigation	10.4	2.0	3.0	3.0	3.0	20.8	31.2	31.2	31.2
	K	Economic opportunities	8.7	4.0	3.0	3.0	4.0	34.8	26.1	26.1	34.8
		TOTAL	100.0	3.0	3.0	3.0	3.0	207.7	220.7	244.0	254.4

Table F- 9. Scoring Result from Profession II

Criteria			Relative Weights(%)	Scores				Weighed Scores			
				1	2	3	4	1	2	3	4
Flood Safety	A	System failure	17.0	1.0	2.0	3.0	4.0	17.0	34.0	51.0	68.0
	B	Flexibility	5.5	1.0	3.0	2.0	2.0	5.5	16.4	10.9	10.9
Construction	C	Materials	2.5	2.0	3.0	3.0	3.0	5.1	7.6	7.6	7.6
	D	Accessibility	5.0	2.0	3.0	3.0	3.0	10.0	14.9	14.9	14.9
	E	Impact on populated area	11.0	2.0	2.0	4.0	4.0	21.9	21.9	43.9	43.9
Ecological impacts	F	Pollution on populated area	11.3	2.0	2.0	2.0	3.0	22.5	22.5	22.5	33.8
	G	Preservation of ecosystem	10.3	2.0	4.0	3.0	2.0	20.5	41.1	30.8	20.5
	H	Salt water intrusion	6.9	4.0	3.0	4.0	4.0	27.5	20.6	27.5	27.5
Economic impacts	I	Protected area&population	10.9	3.0	3.0	4.0	4.0	32.6	32.6	43.4	43.4
	J	Disturbance of navigation	10.4	2.0	3.0	3.0	3.0	20.8	31.2	31.2	31.2
	K	Economic opportunities	8.7	4.0	3.0	3.0	3.0	34.8	26.1	26.1	26.1
		TOTAL	100.0	3.0	3.0	3.0	3.0	218.2	269.0	309.9	327.9

Table F- 10. Scoring Result from a Citizen of Shanghai

Criteria			Relative Weights(%)	Scores				Weighed Scores			
				1	2	3	4	1	2	3	4
Flood Safety	A	System failure	17.0	1.0	1.0	2.0	3.0	17.0	17.0	34.0	51.0
	B	Flexibility	5.5	0.0	1.0	2.0	3.0	0.0	5.5	10.9	16.4
Construction	C	Materials	2.5	3.0	2.0	2.0	2.0	7.6	5.1	5.1	5.1
	D	Accessibility	5.0	4.0	4.0	3.0	2.0	19.9	19.9	14.9	10.0
	E	Impact on populated area	11.0	2.0	1.0	3.0	3.0	21.9	11.0	32.9	32.9
Ecological impacts	F	Pollution on populated area	11.3	2.0	3.0	2.0	2.0	22.5	33.8	22.5	22.5
	G	Preservation of ecosystem	10.3	1.0	1.0	2.0	2.0	10.3	10.3	20.5	20.5
	H	Salt water intrusion	6.9	2.0	2.0	3.0	3.0	13.7	13.7	20.6	20.6
Economic impacts	I	Protected area&population	10.9	2.0	2.0	4.0	3.0	21.7	21.7	43.4	32.6
	J	Disturbance of navigation	10.4	3.0	2.0	3.0	3.0	31.2	20.8	31.2	31.2
	K	Economic opportunities	8.7	4.0	3.0	3.0	4.0	34.8	26.1	26.1	34.8
		TOTAL	100.0					200.7	184.9	262.3	277.6

Appendix G. Boundary conditions

The boundary conditions are described more in detail in this Appendix. The information corresponds to Chapter 6 in the main report.

G.1 Bathymetry

The bathymetry of the protection area is presented in the following figure.

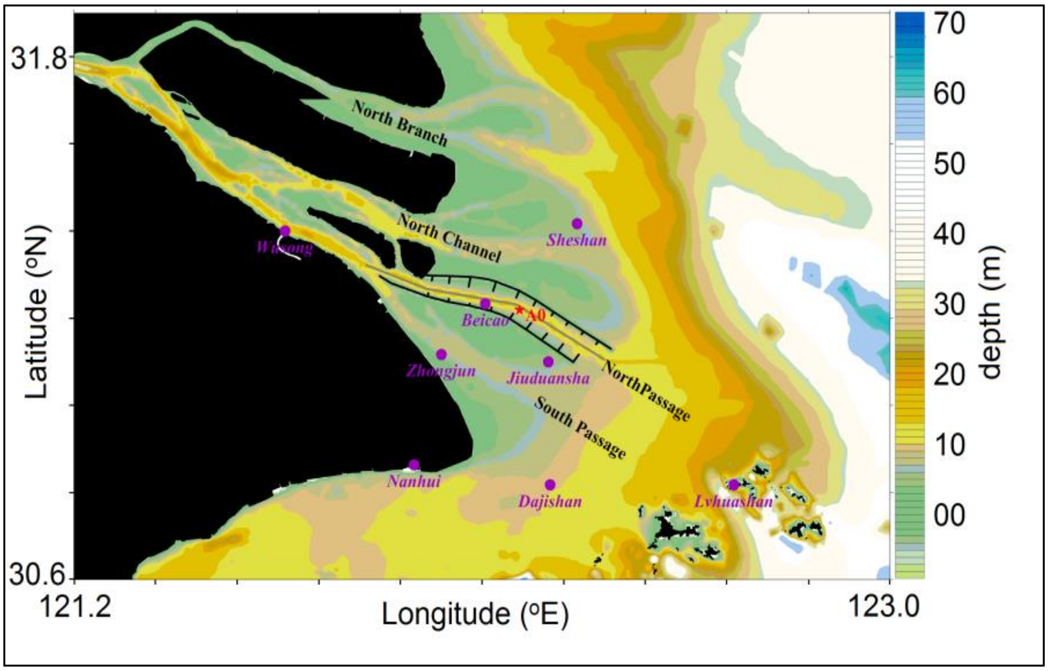


Figure G- 1. Bathymetry of Yangtze Estuary ((X. Wang & Andutta, 2013))

G.2 Hydraulic conditions

- Non-storm hydraulic conditions

  Non-storm hydraulic conditions mean the regular conditions under normal circumstances. These conditions include tidal conditions, sea level rise and salinity.

  Tidal conditions

  The Yangtze Estuary experiences semi-diurnal tides. The mean tidal range is 2.7 m near the mouth. The maximum tidal range is 4.6m.

  Sea level rise

  Sea level rise in the East China Sea has occurred due to several reasons: the global sea level rise, natural subsidence of Yangtze Estuary sediments and the land subsidence caused by excessive groundwater withdrawals. Sea level rise is a relatively large threat to Shanghai flood defense system than other coastal cities in China. Due to global climate change, the absolute sea level change in Wusong is forecasted to rise at a rate of 2.5 mm/a in the period 1999-2030, and continue increasing to 5.0 mm/a in the following two decades. The absolute rising height is 5 cm, 10 cm and 20 cm in 2010, 2030 and 2050(Yin & Xu, 2013).

Discharge

The annual average river discharge through Yangtze Estuary is 29,000 m^3/s. The maximum river discharge is 92,600 m^3/s.

Salinity

Salinity in Yangtze Estuary is around 20-30 parts per thousand. The problem of salty water intrusion is growing due to the decrease of the river discharge, caused by the water level regulation projects and increasing water intake projects in the upstream Yangtze River.

Figure G- 2. Salt Water Intrusion in the Yangtze River Estuary (Source: (Shanghai Water Resource Authourity, 2004))

- Typhoon conditions

Typhoons coming from the western North Pacific (WNP) mostly hit Shanghai during the boreal summer (June to September) every year. This name comes from a Chinese word Tai-fung meaning a great wind from the sea. They always come accompanied by heavy rains.

The storm and wave characteristics are used for a return period of 1/1,000 [1/year]. The maximum water level is MSL+ 6.3m. The wave height is decreased gradually when waves propagates from the out to the inside estuary. The mean wave height is 0.9 m and its period is 3.7 s. The maximum wave height is 5.5 m and the maximum wave period is 12s. According to Goda (1985) the maximum wave height is approximately a factor 1.8 larger than the significant wave height. Thus, the significant wave height is obtained as 3.1 m (Goda, 2010).

Design storm computation

The water level during a design 1/1000 [1/year] storm can be regarded as the superposition of the astronomical tides and storm surges. The tides in the open coast are treated as sinusoidal tides from the sea, with a period of 12h 25minutes:

$$h_{tides} = 1.37\cos(\omega t + \varphi) \tag{G.1}$$

Storm surge can be schematized as:

$$h_{surge} = \begin{cases} hs_{peak} cos(\frac{\pi}{D} t)^2 & if -\frac{D}{2} < t < \frac{D}{2} \\ 0m & otherwise \end{cases} \tag{G.2}$$

Then water levels in the open coast is:

$$h_w = h_{tides} + h_{surge} \tag{G.3}$$

While $hs_{peak} = 4.95m$, $\varphi = 0^o$, D=24h.The computed design storm is shown in

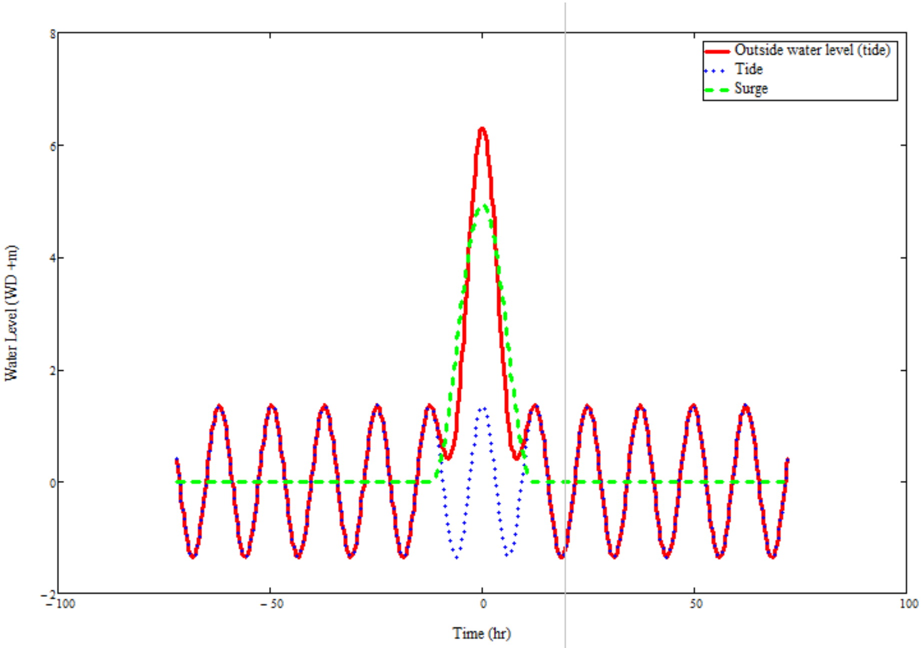


Figure G- 3. Water levels of 1/1,000 [1/year] design storm

G.3 Geotechnical conditions

The geology conditions in the Yangtze Estuary are very complicated. Because of the lack of available data, the geotechnical conditions of the selected barrier location are obtained from some field tests, which are close to the exact barrier location, see Figure G- 4. The schematized subsoil information is indicated in Table G- 1.

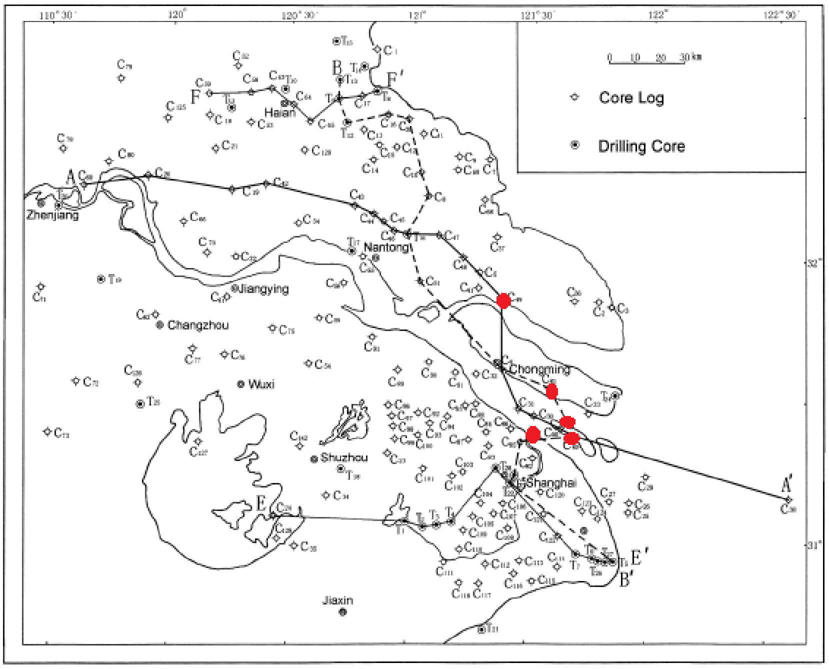


Figure G- 4. Location map of drillholes and stratigraphical sections **(Li et al., 2000)**

Table G- 1. Schematized sub layers in the Yangtze Estuary

Layer	Depth	Thichness	Class	Undrained shear stress	Internal friction angle	Vertical permeability	Young's modulus
	WD (-m)	D (m)		(kPa)	ϕ	P (m/s)	Es (kN/m2)
K1	0~5	5	soft clay	14	25	1.E-09	3000
K2	5~13	8	silty clay	34	30	5.E-09	2500
K3	13~31	18	silt	25	34	5.E-09	11000
K4	31~45	14	firm clay	28	35	1.E-09	3000
K5	45~65	20	silty sand	48	35	8.E-10	14000
K6	below 65		very dense sand				

G.4 Meteorological conditions

The Shanghai city enjoys a subtropical monsoon climate, with obvious features of continental climate. Most precipitation falls during the rainy season from July to September. Annual rainfalls average 1,227 mm.

Table G- 2. Average monthly Main Meteorological Index (Source: Shanghai Meteorological Bureau, 2010)

Month	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec
Prec (mm)	38.8	82.3	103.3	77.3	86.3	93.9	208.8	145.9	248.0	67.3	11.6	63.8

Typhoons moving towards Shanghai not only bring high storm surges but also heavy rainfalls and strong winds to the area. These intense rains temporarily increase the runoff into Huangpu River substantially and may cause flooding of the Huangpu River when this occurs during barrier closure. inside the estuary.

Appendix H. System requirements of design level II

This part describes the requirement of the design level II of Chapter 7 in the main report.

H.1 Nautical consideration

- Vessel traffic characteristics

Before assessing the actual vessel traffic on the North Channel, the main inland vessel characteristics are discussed. By European standard, navigation requirements are completely defined by specifying the river class. PIANC (Permanent International Association of Navigation Congress) developed a worldwide used classification of vessels, including associated waterways.

It is important to note that the maximum clearance height of all the larger commercial vessels is 9.1 m. This clearance height can be adopted as the design criteria.

Table H- 1. Authorized Navigation Channel in the Yangtze River Estuary

Channel Location	Channel type	Dimension of channel		Design Vessel					
		Width (m)	Depth (m)	Vessel type			Total length (m)	Beam Width (m)	Laden draft (m)
North Passage	Main navigation channel	350	12.5	ocean-going vessels	all weather conditions	4th generation container ship	275	32.2	12.5
					sailing with the tide	5th and 6th container ship	280	39.8	14
						100,000 DWT bulk ship	260	39	15.2
South Passage	Assistant navigation channel	250	8	smal vessels and shallow draft vessels going along the south coast	sailing with the tide	10,000 DWT bulk ship	130	20.5	8.5
North Channel	Assistant navigation channel	300	10	small vessels going along the north coast	sailing with the tide	10,000 DWT bulk ship	130	20.5	8.5
North Branch	under development	100	5	small shallow draft vessels among the nearby small cities					

- Navigation channel dimensions

The navigational requirements are set according to a report “Approach Channel – A Guide for Design” by PIANC and “Waterway Guideline 2011” released by Rijkswaterstaat.

With the type and dimensions of the design ship chosen, the preliminary design of the navigation channel can be chosen.

<u>Width Consideration</u>

For the channel width design in straight sections, the bottom width w of the waterway,

is given for one-way channel by:

$$W = W_{BM} + \sum_{i=1}^{n} W_i + W_B \qquad \text{(H.1)}$$

and for two-way channel by:

$$W = 2W_{BM} + 2\sum_{i=1}^{n} W_i + W_B + \sum W_p \tag{H.2}$$

Where,

- W_{BM}, basic manoeuvring lane, for moderate channels, W_{BM}=1.5 B (design vessel beam width) , [m]
- W_i, additional width, [m]
- W_B, bank clearance, [m]
- W_p, passing distance, [m]

Depth Consideration

The navigation channel depth is estimated from:

- At rest draught of design vessel
- Tide height through transit of the channel
- Squat (vessel motion)
- Wave-induced motion
- A margin depending on type of bottom
- Water density and its effect on draught.

As the detailed information is absent, a simple approach is adopted. There is a minimum water depth required to enable ships to pass through the approach channel and lock. For maritime vessels a rule of thumb for the water of depth D_w would be:

$$D_w = D_S \times 1.15 + 0.5 \tag{H.3}$$

Where D_S is the loaded draft of the design vessel.

The reference to design vessel is defined as:

- Beam B_S : distance between port side to starboard side
- Length L_S : distance between stern and bow at ships
- Draught D_S : distance between the undersides of the deck amidships to the keel's bottom
- Results

South Channel

Table H- 2. Nautical requirements in the South Channel

Design vessel			
Beam width	Bs	32.5	m
Length	Ls	275	m
Draught	Ds	12.5	m

Channel width consideration			
Basic manoeuvring lane		1.5	B
Additional width	Addition for speed	0	B
	Addition for cross wind	0.3	B
	Addition for cross current	0.1	B
	Addition for longitudinal curren	0	B
	Addition for waves	0	B
	Addition for aids to navigation	0.1	B
	Addition for bottom surface	0	B
	Addition for cargo hazard	0.1	B
Bank clearance		0.6	B

Table H- 3. Navigation channel dimensions in the South Channel

	Width	unit	Deepth	unit
One-way	2.7	Bs	1.3	Ds
	87.75	m	16.25	m
Two-way	5.3	Bs	1.3	Ds
	172.25	m	16.25	m

North Channel

Table H- 4. Nautical requirements in the North Channel

Design vessel			
Beam width	Bs	20.5	m
Length	Ls	130	m
Draught	Ds	8.5	m

Channel width consideration			
Basic manoeuvring lane		1.5	B
Additional width	Addition for speed	0	B
	Addition for cross wind	0.3	B
	Addition for cross current	0.1	B
	Addition for longitudinal curren	0	B
	Addition for waves	0	B
	Addition for aids to navigation	0.1	B
	Addition for bottom surface	0	B
	Addition for cargo hazard	0.1	B
Bank clearance		0.6	B

Table H- 5. Navigation channel dimensions in the North Channel

	Width	unit	Deepth	unit
One-way	2.7	Bs	1.3	Ds
	55.35	m	11.05	m
Two-way	5.3	Bs	1.3	Ds
	108.65	m	11.05	m

North Branch

North Branch is seldom used as a navigation channel. Only in some particular circumstances there would be some small fishery boats passing through the North Branch. In that case, a small opening is left for local people. The channel profile is designed with a width of 60 meters and a depth of 5 meters.

- Check on traffic intensity

The average day would enjoy 150 vessels for two-way traffic. This means 75 vessels in each direction on average. For a quick calculation all of the vessels have the same size and 12 hours service is applied. This means a vessel would take $\frac{12*3600}{75} = 576$ seconds to pass the channel. The limit speed of design vessel can be calculated according to Groeneveld:

$$\left(\frac{v_{lim}}{\sqrt{gd_{nav}}}\right) = 0.78 \times (1 - \frac{A_s}{A_c})^{2.25} \qquad \text{(H.4)}$$

Where,

v_{lim}: Limit sailing velocity of design vessel [m/s]

A_s: Wet surface of design vessel [m^2]

A_c: Wet surface of navigational cross section [m^2]

d_{nav}: Depth of navigational channel [m]

L_s : Length of design vessel [m]

The vessels will be travelling at half the limit speed, so every vessel needs 576*0.5*V_{lim} of space. The minimum mutual distance is 1.45 L_s. That means the minimum required space for each vessel is 1.45 L_s + L_s. The minimum required length must be smaller than the available space for each vessel. The results of each navigation channel are shown in Table H- 6. It shows all the designed channels are sufficient. No additional barge lanes are required.

Table H- 6. Traffic intensity check for each navigational channel

		Ac (m^2)	As (m^2)	Limit speed (m/s)	Available space (m)	Minimum required length (m)
North Channel	One-way	916	174	5.05	1454	319
	Two-way	1505	174	6.15	1772	319
South Channel	One-way	2081	406	5.3	1527	673
	Two-way	3459	406	6.52	1877	673
North Branch	Only kept for small fishery boats -> sufficient space					

H.2 Environmental consideration

When considering the environmental aspect of the barrier, an awareness in many people of the need to protect this area's natural resources and unique tidal habitat must be taken in to account. A main environmental requirement is set as the barrier must allow the tides to enter freely, thus maintaining the tidal ecosystem.

<u>Step 1: Model set-up (discrete system)</u>

Yangtze River Estuary is the part of the Yangtze River system downstream from Datong where the tidal limit is. The estuary has an approximate length of 630 km and a mouth of about 90 km width. The reach upstream from the junction (113 km from the estuary mouth) of the North Branch and the South Branch to Datong is called "the Upper Reach", the downstream part is called "the Lower Reach" or "the River Mouth".

In reality, the discharge and the surface elevation both in and out of the basin (estuary) are always treated as continuous functions of distance and time. Normally, in a simplified approach, certain flow system can be schematized in terms of separate but connected basins of finite dimensions. In this case, a discrete model is set up. The disregard of special variations in the basins considered is allowed if the dimensions are small compared to a typical length of the long waves in the domain. With a mean water depth of between 10- 15 meters and a tidal period of around 12 hours, the tidal wave length for the Yangtze River Estuary basin lies between 450 km and 550 km. Since the actual basin is 113 km, which is much less than the tidal wave length. In such case, phase differences within the system are negligible. Started another way, the flow motion losses its wave-like character.

A model is set up as a discrete system. *This theory applies to such systems consisting of a nearly closed basin or a reservoir, connected through soma narrow, short opening or a channel of some length to an external body of water with a time-varying water level.* The basin is relatively small compared to the tidal wave length, and it is partially closed except for a connection to the seawater. The assumption "small basin" can be applied here, in which situation the flow velocities in the estuary are quite low. Flow resistance and inertia are negligible. Therefore, the water level can be assumed to be horizontal all the times. The water level in the basin is assumed to be horizontal at all times, varying in time only. Its variation is modeled with an ordinary differential equation (ODE) instead of a partially differential equation (PDE). The opening of the barrier system can be viewed as the inlet or gap, connecting the tidal sea to the basin. In this system, the only function of the basin is storage; its connection to the sea has an only function of transport.

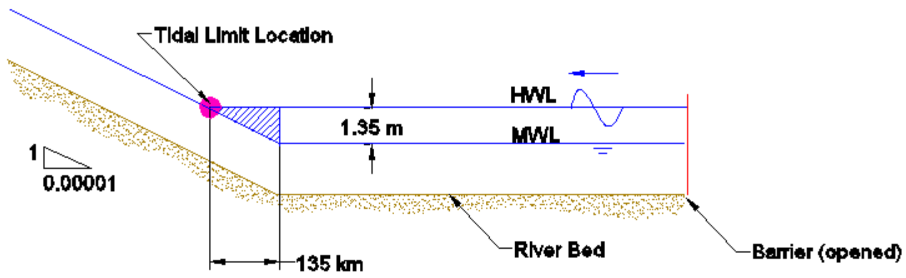


Figure H- 1. Indication of idealized tidal limit in the Yangtze River

Figure H- 2. Map of idealized tidal limit in the Yangtze River

However, according to this theory, the resistance in the estuary is neglected and the influence of the upstream river is not included. Considering the 8 km wide river mouth connecting the Yangtze River to the estuary, an additional storage (shadow area in Figure H- 2) should be added to the basin, if the above theory is applied in the system. The total storage area in indicated in Figure H- 3.

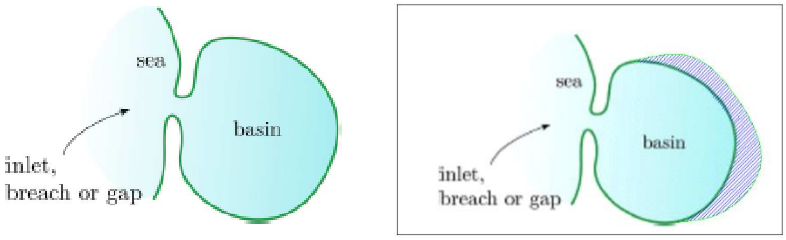


Figure H- 3. Lift: original discrete system; Right: modified Discrete System with additional storage area

Step 2: Non-storm conditions

The tides in the sea are semidiurnal; the tides can be characterized as sinusoidal:

$$\xi_s = \tilde{\xi_s} \cos \omega t \tag{H.5}$$

Since the response is also sinusoidal, with the same frequency, it can be rewritten as

$$\xi_b = \tilde{\xi_b} \cos(\omega t - \theta) = r\tilde{\xi_s} \cos(\omega t - \theta) \tag{H.6}$$

Where,

- $\tilde{\xi_s}$ is the tidal amplitude in the sea [m]
- $\tilde{\xi_b}$ is the tidal amplitude in the basin [m]
- $r = \tilde{\xi_s}/\tilde{\xi_b}$, is the ration between the two amplitude [-]
- θ is the phase lag of the water level in the basin [rad]
- ω is the wave frequency $\omega = 2\pi/T$ [s^{-1}]
- T is the wave period, (~ 12h)

$$\Gamma = \frac{8}{3\pi}\,\chi\,(\frac{A_b}{A_c})^2\,\frac{\omega^2\widetilde{\xi_s}}{g} \tag{H.7}$$

- Γ is the dimensionless parameter, containing all independent variables playing a role in the present problem, [-]
- A_b is the surface area of the basin, [=1.8* 10 ^{9}m^2]
- A_c is the wet area of the opening [m^2]
- g is the gravitational acceleration [=9.8 m/s^2]
- χ is the dimensionless loss coefficient, [=0.5]

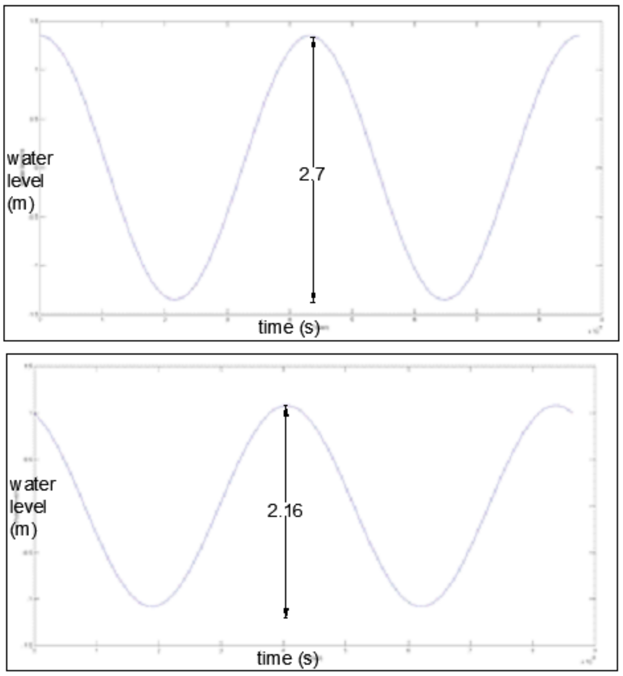


Figure H- 4. Schematized tidal propagation in the sea (upper); tidal propagation in the estuary (lower)

Another relationship between r and Γ can be written as:

$$r = \cos\theta = \frac{1}{\sqrt{2\Gamma}}\sqrt{-1+\sqrt{1+4\Gamma^2}} \tag{H.8}$$

With,

$\widetilde{\xi_s}$	1.35	meters
$\widetilde{\xi_b}$	1.08	meters
T	12	hours
r	0.8	
A_b	1.8 *10^9	m^2

Because the Yangtze River Estuary is a sensitive hydrodynamic system, but the habitats there are not that outstanding compared to the Scheldt estuary in the Netherlands. The criteria is set as r=80%, referring to 87% in the design of Easter Scheldt barrier. In other word, after tides pass through the barrier, the tide amplitude is reduced to 80% of their original shape, due to the effects of the constriction of the river mouth.

Substituting the variants into the formula, the maximum wet area of the opening is obtained as A_c= 108,000 m^2.

The closure of the Yangtze River Estuary is calculated as

$$1 - \frac{108{,}000}{15000 \times 12.5} = 42.4\ \%$$

In a study by Ruijs (2011), the impact of a partial closure on the Galveston Bay's hydrodynamics has examined. He stated a maximum 40% reduction of the flow area is suitable for the design in the Bolivar Roads barrier. As a conservative estimate, a constriction of 40% is used in this thesis.

The whole wet area of the environmental section:

$$Y = 15{,}000 \times 12.5 \times 60\% - X_1 - X_2 - X_3 = 105{,}616\ m^2$$

*X_1, X_2, X_3 are the wet area of the three navigation channels, they are 3,520 m^2, 3,001 m^2 and 363 m^2. Assuming the average dredged depth is 12.5 meters, the length of environmental section is around 8.5 km.

Appendix I. Influence on the landside area

The obvious propose of the barrier is to protect Shanghai and inner cities from coastal flooding due to storm surges. One of the potential disadvantages of the barrier is that it may cause flooding on the landward side of the barrier as the Yangtze River water levels are high or the barrier is closed for an extended period. River flooding occurs as a result of the accumulation of the high river discharge due to the obstruction of the continued river discharge by the barrier.

This should not present a problem to the upstream cities, such as Nanjing, Changzhou and Suzhou, which are 200 km, 150 km, 50 km from Shanghai, respectively. The maximum duration of closure should be undertaken seriously. The problem can be solves in careful decision on two aspects: when and how long the barrier should be closed. When an extreme is expected, the barrier should be closed in advance. If the barrier is closed during low tides, there would be more storage area to sustain the upstream water. Hence, the discussion is based on the type of flooding events and weather the barrier is closed at high/low tide:

- The surge tide coinciding with Yangtze River flooding (barrier closed at high tide)
- The surge tide coinciding with Yangtze River flooding (barrier closed at low tide)

Table I- 1. Current situations of the Nanjing, Changzhou, Suzhou and Shanghai

	City	Population (million)	Distance from the Yangtze Estuary (km)	Max. Water level (CD)
B	Nanjing	6.5	200	11.2 m
C	Changzhou	3.5	100	11.0 m
D	Suzhou	1.3	50	11.0 m
A	Shanghai	14	0	11.5 m

The water level in the estuary during the barrier closure becomes:

$$H_b = H_b, o + \frac{Q \times \Delta T}{A} \qquad \text{(H.9)}$$

Where,

H_b, o: Original water level in the estuary before barrier closure [m]

H_b: Water level in the estuary during barrier closure, [m]

A: Surface area in the estuary,A=881 km^2

ΔT: Barrier closure time [s]

Q: Yangtze river discharge [m^3/s]

The backwater curve can be calculated using following equations:

$$h_i = h_e + (h_b - h_e)(\frac{1}{2})^{-\frac{x-x_0}{L_{1/2}}} \tag{H.10}$$

In which the boundary condition $h=h_b$ at $x=x_0$ has been used and the so –called 'half- length' $L_{1/2}$ is given by:

$$L_{1/2} = \frac{0.24}{i_b}(\frac{h_b}{h_e})^{4/3} \tag{H.11}$$

Where,

h_c: Critical depth, $h_c = (\frac{q^2}{g})^{1/3}$,[m]

h_e: Equivalent depth. $h_e = (\frac{q^2}{i_b C^2})^{1/3}$,[m]

C: Chezy coefficient, in index for bed roughness, for Yangtze River, C= 50

i_b: Bed slope, $1*10^{-5}$

q: unit discharge, q=Q/B, B=8km

The point in the surge cycle at which all the gates close will determine the water levels inside and outside of barrier. The barrier can be closed at high or low tides, resulting in different water levels in the reservoir. The results show, if the barrier is closed at high tide, the local levees along the Yangtze River in Nanjing (city B) cannot sustain such high water level, even though the closure period is reduced to 8 hours, which is already not sufficient to protect against the coastal flooding induced by storm surges. In the opposite case, if the barrier is closed at low tide, which provides higher storage capacity of the estuary, with the maximum discharge comes from the Yangtze River, those cities can still remain safe even the estuary is blocked off for 16 hours. Results refer to

Table I- 2 and

Table I- 3.

Table I- 2. Surface profile of Yangtze River at $Q=Q_{max}$ when the barrier is closed at high tide

HIGH Q+HIGH TIDE, T=12H												
City	Q (m^3/s)	h0 (m)	T (hour)	I	he (m)	x (m)	L1/2 (m)	Depth (m)	elevation	:er level (CD	vee elevation (CD r	SUFFICIENT
B	92600.00	18.42	12.00		17.50	200000.00	449717.83	18.75	-5.00	13.75	11.20	N
C	92600.00	18.42	12		17.50	100000.00	449717.83	18.57	-6.00	12.57	11.00	N
D	92600.00	18.42	12.00		17.50	50000.00	449717.83	18.49	-6.50	11.99	11.00	N
A	92600.00	18.42	12.00		17.50	0.00	449717.83	18.42	-7.00	11.42	11.50	N
HIGH Q+HIGH TIDE, T=11H												
City	Q (m^3/s)	h0 (m)	T (hour)	I	he (m)	x (m)	L1/2 (m)	Depth (m)	elevation	:er level (CD	vee elevation (CD r	SUFFICIENT
B	92600.00	18.04	11.00		17.50	200000.00	437443.01	18.24	-5.00	13.24	11.20	N
C	92600.00	18.04	11.00		17.50	100000.00	437443.01	18.14	-6.00	12.14	11.00	N
D	92600.00	18.04	11.00		17.50	50000.00	437443.01	18.09	-6.50	11.59	11.00	N
A	92600.00	18.04	11.00		17.50	0.00	437443.01	18.04	-7.00	11.04	11.50	Y
HIGH Q+HIGH TIDE, T=10H												
City	Q (m^3/s)	h0 (m)	T (hour)	I	he (m)	x (m)	L1/2 (m)	Depth (m)	elevation	:er level (CD	vee elevation (CD r	SUFFICIENT
B	92600.00	17.66	10.00		17.50	200000.00	425253.71	17.73	-5.00	12.73	11.20	N
C	92600.00	17.66	10.00		17.50	100000.00	425253.71	17.69	-6.00	11.69	11.00	N
D	92600.00	17.66	10.00		17.50	50000.00	425253.71	17.68	-6.50	11.18	11.00	N
A	92600.00	17.66	10.00		17.50	0.00	425253.71	17.66	-7.00	10.66	11.50	Y
HIGH Q+HIGH TIDE, T=9H												
City	Q (m^3/s)	h0 (m)	T (hour)	I	he (m)	x (m)	L1/2 (m)	Depth (m)	elevation	:er level (CD	vee elevation (CD r	SUFFICIENT
B	92600.00	17.29	9.00		17.50	200000.00	413151.15	17.20	-5.00	12.20	11.20	N
C	92600.00	17.29	9.00		17.50	100000.00	413151.15	17.25	-6.00	11.25	11.00	N
D	92600.00	17.29	9.00		17.50	50000.00	413151.15	17.27	-6.50	10.77	11.00	N
A	92600.00	17.29	9.00		17.50	0.00	413151.15	17.29	-7.00	10.29	11.50	Y
HIGH Q+HIGH TIDE, T=8H												
City	Q (m^3/s)	h0 (m)	T (hour)	I	he (m)	x (m)	L1/2 (m)	Depth (m)	elevation	:er level (CD	vee elevation (CD r	SUFFICIENT
B	92600.00	16.91	8.00		17.50	200000.00	401136.57	16.66	-5.00	11.66	11.20	N
C	92600.00	16.91	8.00		17.50	100000.00	401136.57	16.80	-6.00	10.80	11.00	N
D	92600.00	16.91	8.00		17.50	50000.00	401136.57	16.85	-6.50	10.35	11.00	N
A	92600.00	16.91	8.00		17.50	0.00	401136.57	16.91	-7.00	9.91	11.50	Y

Table I- 3.Surface profile of Yangtze River at $Q=Q_{max}$ when the barrier is closed at low tide

HIGH Q+LOW TIDE, T=12H											
City	Q (m^3/s)	q	h0 (m)	T (hour)	he (m)	x (m)	L1/2 (m)	Depth (m)	'ater level (CD	r elevation (	SUFFICIENT
B	92600.00	11.58	15.66	12.00	17.50	200000.00	362198.46	14.80	9.80	11.20	Y
C	92600.00	11.58	15.66	12.00	17.50	100000.00	362198.46	15.27	9.27	11.00	Y
D	92600.00	11.58	15.66	12.00	17.50	50000.00	362198.46	15.48	8.98	11.00	Y
A	92600.00	11.58	15.66	12.00	17.50	0.00	362198.46	15.66	8.66	11.50	Y
HIGH Q+LOW TIDE, T=13H											
City	Q (m^3/s)	q	h0 (m)	T (hour)	he (m)	x (m)	L1/2 (m)	Depth (m)	'ater level (CD	r elevation (	SUFFICIENT
B	92600.00	11.58	16.04	13.00	17.50	200000.00	373913.64	15.38	10.38	11.20	Y
C	92600.00	11.58	16.04	13.00	17.50	100000.00	373913.64	15.74	9.74	11.00	Y
D	92600.00	11.58	16.04	13.00	17.50	50000.00	373913.64	15.90	9.40	11.00	Y
A	92600.00	11.58	16.04	13.00	17.50	0.00	373913.64	16.04	9.04	11.50	Y
HIGH Q+LOW TIDE, T=14H											
City	Q (m^3/s)	q	h0 (m)	T (hour)	he (m)	x (m)	L1/2 (m)	Depth (m)	'ater level (CD	r elevation (	SUFFICIENT
B	92600.00	11.58	16.42	14.00	17.50	200000.00	385721.32	15.95	10.95	11.20	Y
C	92600.00	11.58	16.42	14.00	17.50	100000.00	385721.32	16.20	10.20	11.00	Y
D	92600.00	11.58	16.42	14.00	17.50	50000.00	385721.32	16.32	9.82	11.00	Y
A	92600.00	11.58	16.42	14.00	17.50	0.00	385721.32	16.42	9.42	11.50	Y
HIGH Q+LOW TIDE, T=14,5H											
City	Q (m^3/s)	q	h0 (m)	T (hour)	he (m)	x (m)	L1/2 (m)	Depth (m)	'ater level (CD	r elevation (	SUFFICIENT
B	92600.00	11.58	16.61	14.50	17.50	200000.00	391659.39	16.23	11.23	11.20	Y
C	92600.00	11.58	16.61	14.50	17.50	100000.00	391659.39	16.43	10.43	11.00	Y
D	92600.00	11.58	16.61	14.50	17.50	50000.00	391659.39	16.52	10.02	11.00	Y
A	92600.00	11.58	16.61	14.50	17.50	0.00	391659.39	16.61	9.61	11.50	Y
HIGH Q+LOW TIDE, T=15H											
City	Q (m^3/s)	q	h0 (m)	T (hour)	he (m)	x (m)	L1/2 (m)	Depth (m)	'ater level (CD	r elevation (	SUFFICIENT
B	92600.00	11.58	16.61	14.50	17.50	200000.00	391659.39	16.23	11.23	11.20	Y
C	92600.00	11.58	16.61	14.50	17.50	100000.00	391659.39	16.43	10.43	11.00	Y
D	92600.00	11.58	16.61	14.50	17.50	50000.00	391659.39	16.52	10.02	11.00	Y
A	92600.00	11.58	16.61	14.50	17.50	0.00	391659.39	16.61	9.61	11.50	Y

Appendix J. Wind set-up

Storm surge level in the estuary is dependent on the wind brought by typhoon. The water level in the estuary is influenced by the water depth, which will be increased when a storm hits. Wind set-up is the increase of water level regarding to the astronomical tide, as the subtraction of astronomical tide from observed water level. The effects on surge levels by wind set up in the water domain is determined by its length and the wind speed blowing over the domain. The Yangtze Estuary is modeled as a circular closed basin, with a fetch of 50km and average depth of 12.5 meters.

The wind set-up can be simply calculated by equation (Lansen & Kluyver, 2006) :

$$\Delta h_{wind} = \frac{1}{2}\frac{\rho_{air}}{\rho_w} C_D \frac{U_{10}^2}{gh} F \qquad (J.1)$$

Where,

Δh_{wind}: Maximum wind set-up [m]

U_{10}: Wind speed at an elevation of 10 above MSL brought by a super typhoon[3] [=65 m/s]

ρ_{air}: Mass density of air [=0.0121 kg/m^3]

ρ_w : Mass density of water [=10.25 kg/m^3]

C_D: Air/ water drag coefficient [=0.0008 – 0.003]

g: Gravitational acceleration [=10 m/s^2]

h: Average water depth [=12.5 m]

F: Length of closed domain [=22000 m]

According to the Shanghai Water Authority, the maximum allowed surge level in the Yangtze Estuary is WD +4.8m. When the surge level arrived to WD+4.8m, the corresponding water level is increased by 3.5m, measured by the above equation.

[3] Wind speed for a "super-typhoon" is representative for a 1/1,000 [1/year] storm. It is defined by the U.S. Joint Typhoon Warning Center for typhoons, which have maximum sustained 1-minute surface winds of at least 65 m/s (130 kt).

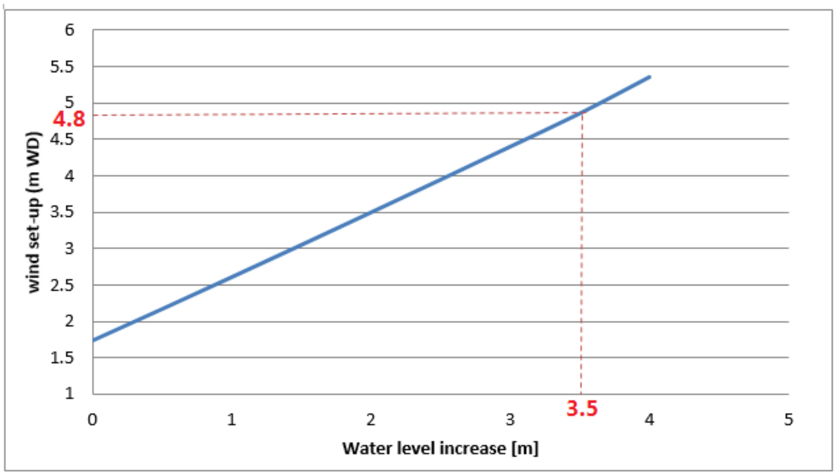


Figure J- 1. Maximum wind set-up related to water level increase inside the estuary

Note: This number determines much for the required retaining height of barriers. More advanced modelling software is recommended to model a storm to calculate the surge levels more accurately. Additionally, economic considerations should be taken into account, including the acceptable damage in the flood prone area, to determine the allowed surge level in the estuary. These are out of scope in this study.

Appendix K. Barrier system design in the Yangtze Estuary

K.1 Barrier system alternatives

This part describes the proposed alternatives for the barrier system in the Yangtze Estuary.

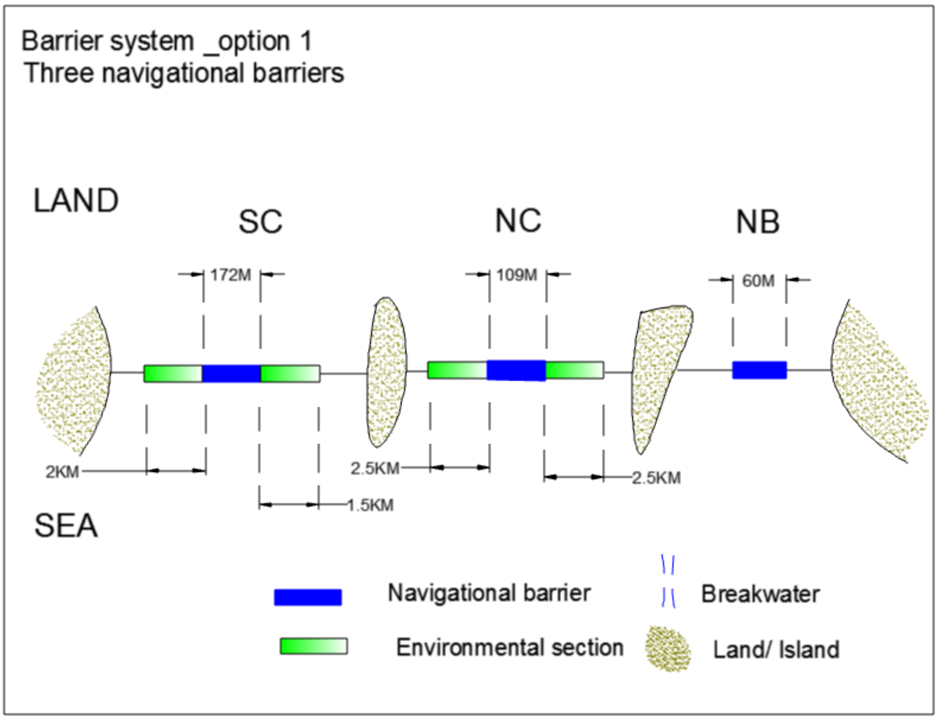


Figure K- 1. Barrier system option 1 – three navigational barriers

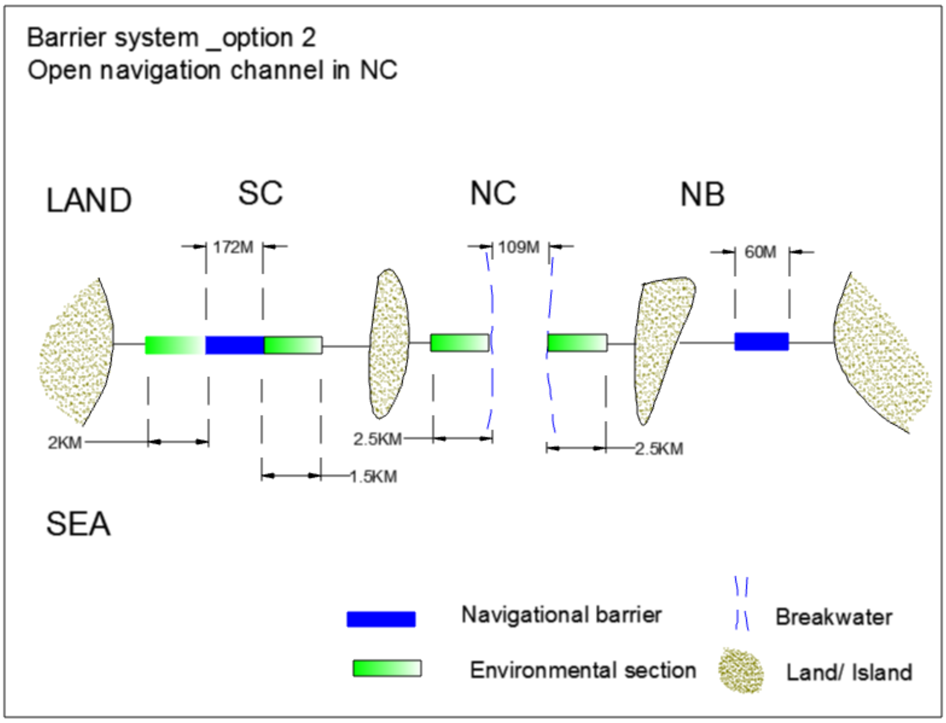


Figure K- 2. Barrier system option 2 – open navigation channel in North Channel

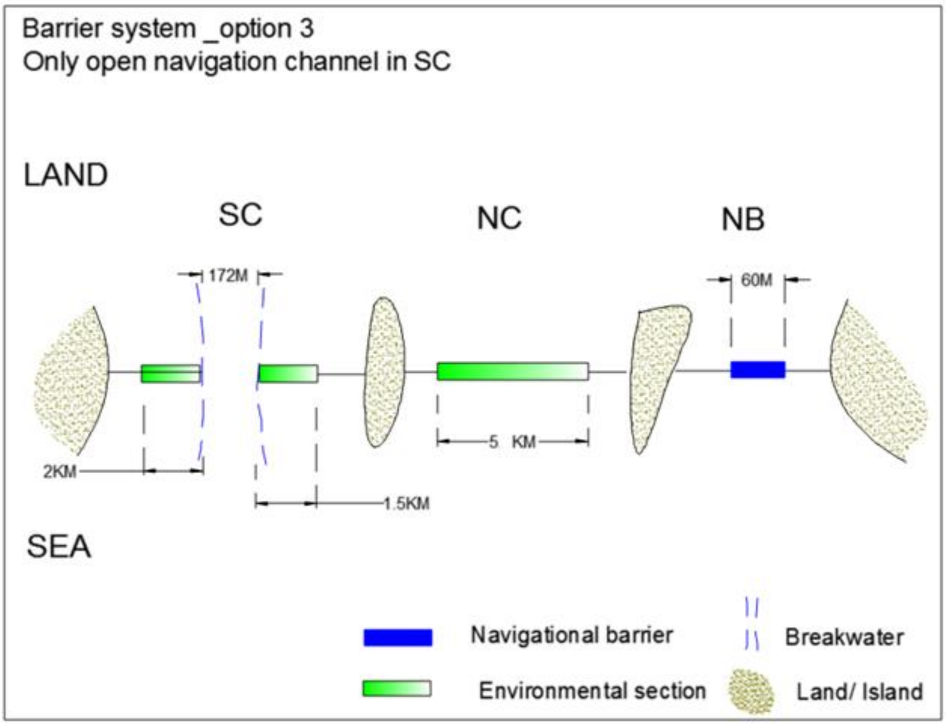


Figure K- 3. Barrier system option 3 – only open navigation channel in South Channel

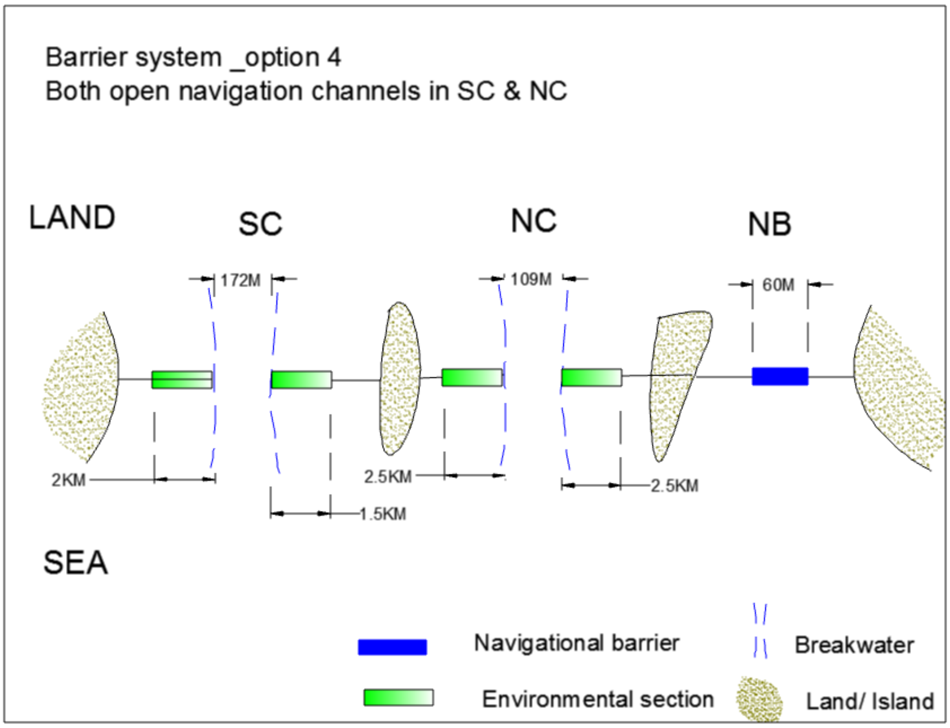


Figure K- 4. Barrier system option 4 – both open navigation channels in South Channel and North Channel

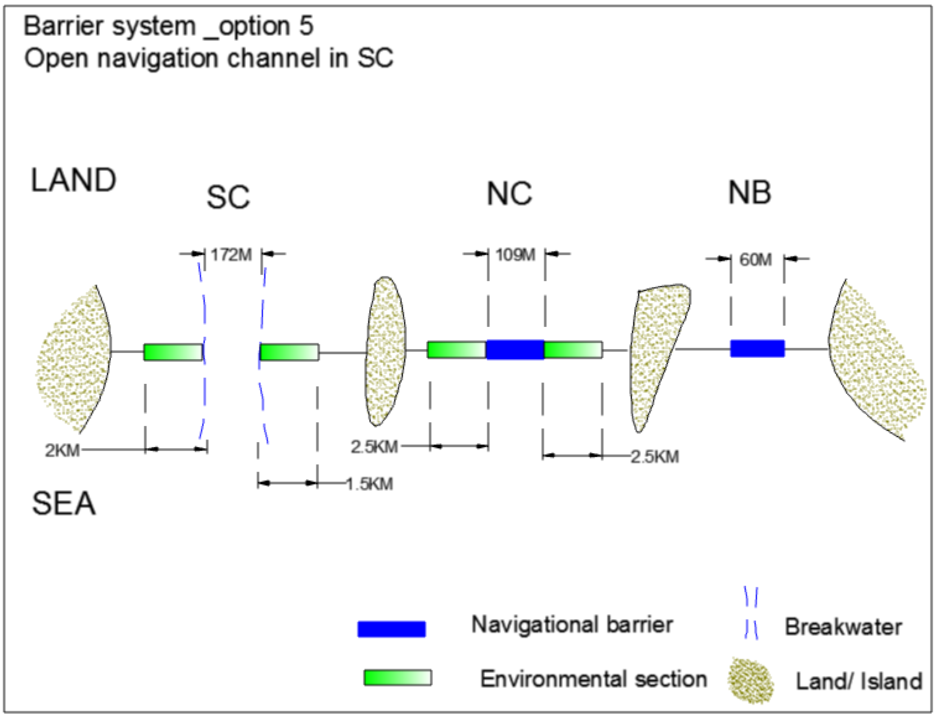


Figure K- 5. Barrier system option 5 – open navigation channel in South Channel

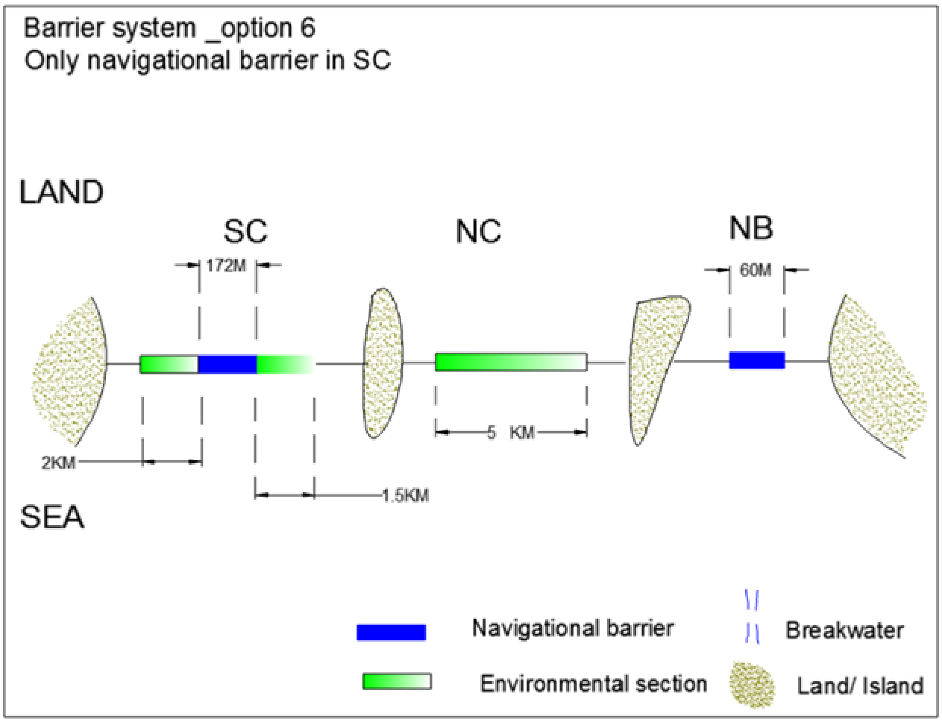


Figure K- 6. Barrier system option 6 – only navigational barrier in South Channel

K.2 Open navigation channel

The first step is to model the current situation. The navigation section is indicated in Figure K-7. It is composed of a channel with length L=1500 m, depth d=16.25m, width B=172m. Straight breakwaters at both sides protect the channel. The bottom is protected with large stones. The navigational section is kept open during storm, while the environmental section can fully block the storm surges.

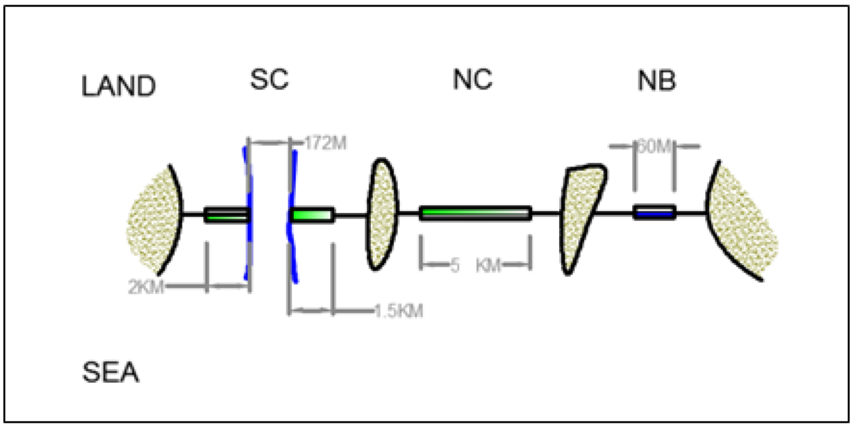


Figure K- 7. Plan view of open navigation channel option

As an assumption, the 1/1,000 [1/year] design storm is released right in front of the barrier. The storm duration is long enough to let the flow spread out the whole estuary.

<u>Method 1: Discrete system</u>

A model is set up as a discrete system. Different to calculation on environmental section, the "rigid column approximation" is applied(Labeur, 2007). The channel length is relatively short compared to the surge length with the storage of the channel is negligible compared to the basin.

Secondly, according to Labeur (2007), the water levels in a semi-enclosed basin that is connected to the sea by a straight, short channel can be calculated based on formula:

$$h_{basin} = h_{coast} - \left(\frac{1}{2} + \frac{c_f L_{channel}}{R_c}\right)\frac{|Q_c|Q_c}{gA_C^2} \tag{K.1}$$

With,

h_{basin}: Water level inside basin [m]

h_{coast}: Water level in open coast [m]

c_f: Friction coefficient

$L_{channel}$: Channel length [=1500m]

R_c: Hydraulic radius [m], $R_C = \frac{A_C}{2d+B}$

Q_c: Discharge through channel [m^3/s]

A_C : Flow area of channel [m^2]

The friction coefficient can be calculated using the following formula (Zhou, 2001):

$$c_f = \frac{g}{C^2} \tag{K.2}$$

$$\mathrm{C} = 18\log(\frac{12d}{Hripple}) \tag{K.3}$$

While H_{ripple} is the height of bed ripples. In this open option, the channel bottom are to be protected by large stones in order to increase bottom roughness. In the preliminary design phase, the stones of $D_{90=}$ 1m is selected, thus the ripples height is around o.1 m under normal circumstances.

However, the above equations contain two unknown variables: h_{basin} and Q_c. An additional equation is required to solve this system:

$$h_{basin}(i+1) = h_{basin}(i) + Q_c(i) \cdot \frac{\Delta \mathrm{t}}{A_{basin}} \tag{K.4}$$

With,

$h_{basin}(i+1)$: Water level inside the estuary, at t = i+1, [m]

$h_{basin}(i)$: Water level inside the estuary, at t = i, [m]

$Q_c(i)$: Discharge through the channel, at t = I [m^3/s]

Δt: Time interval between datapoints (= 900 s)

The discharge through the channel (at t = i) can be calculated according to equation:

$$Q_c = -(h_{basin} - h_{coast}) \cdot A_{barrier}\sqrt{2g(h_{basin} - h_{coast})} \quad +Q_{river} \tag{K.5}$$

By using all these equations, the water levels inside the estuary in the open option can be calculated. The results are shown in following figure. The maximum water level is WD+ 3.1 m, which is smaller than the allowable water level WD + 3.5m[4].

[4] The maximum allowed surge level inside the estuary is WD+ 4.8m. A surge of WD+ 4.8 m occurs when the water level in the estuary has increased to WD +3.5m.

Method 2: Math CAD calculation sheet

Assignment: size of storage basin and discharge facilities

$ORIGIN := 1$

Characteristics of incoming tides

$T := 12 \cdot hr + 25 \cdot min$ $\quad \omega := \frac{2 \cdot \pi}{T}$

$h_{mx} := 1.35 \cdot m$

$h_{mn} := -1.35 \cdot m$

$A := \frac{h_{mx} - h_{mn}}{2}$ $\quad A = 1.35\,m$ (amplitude of incoming tide)

$h_{mid} := \frac{h_{mx} + h_{mn}}{2}$ $\quad h_{mid} = 0\,m$

$\varphi := 0 \cdot \pi$ (phase lag between incoming tide and tide in the basin)

$h_{tide}(t, \Delta) := h_{mid} + \Delta + A \cdot \cos(\omega \cdot t + \varphi)$

$$h_{surge}(t, D, h_{s\_peak}) := \begin{cases} \left(h_{s\_peak} \cdot \cos\left(\frac{\pi}{D} \cdot t\right)^2\right) & \text{if } \frac{-D}{2} < t < \frac{D}{2} \\ (0 \cdot m) & \text{otherwise} \end{cases}$$

$h_w(t, \Delta, D, h_{s\_peak}) := h_{tide}(t, \Delta) + h_{surge}(t, D, h_{s\_peak})$

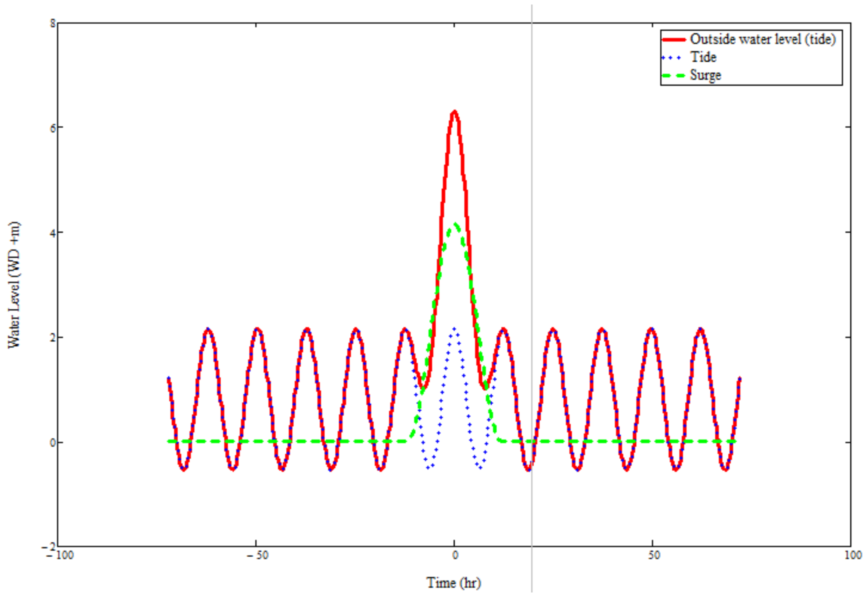
$h_{s\_peak} := 4.15 \cdot m$ (max.surge level) $\quad D := 24 \cdot hr$ (Design surge period)

$t := -3D, -3D + 15 \cdot min .. 3D$

$\Delta := 0.8m$ (sea level rise and land subsidence in 100 years)

$A_{OSK} := 1800\,km^2$ (surface area of estuary)

$Q_{riv} := 29000 \frac{m^3}{s}$ (discharge of Yangtze River)



$$h_{in} := 0 \cdot m$$

$$Q_{uit\_fnc}(A_{barr}, h_{in}, h_{uit}) := -\mathrm{sign}(h_{in} - h_{uit}) \cdot A_{barr} \cdot \sqrt{2 \cdot g \cdot |h_{in} - h_{uit}|}$$

$$dh\_dx(t, h_{in}, A_{barr}, A_{OSK}, \Delta, D, h_{s\_peak}) := \frac{Q_{uit\_fnc}(A_{barr}, h_{in}, h_w(t, \Delta, D, h_{s\_peak})) + Q_{riv}}{A_{OSK}}$$

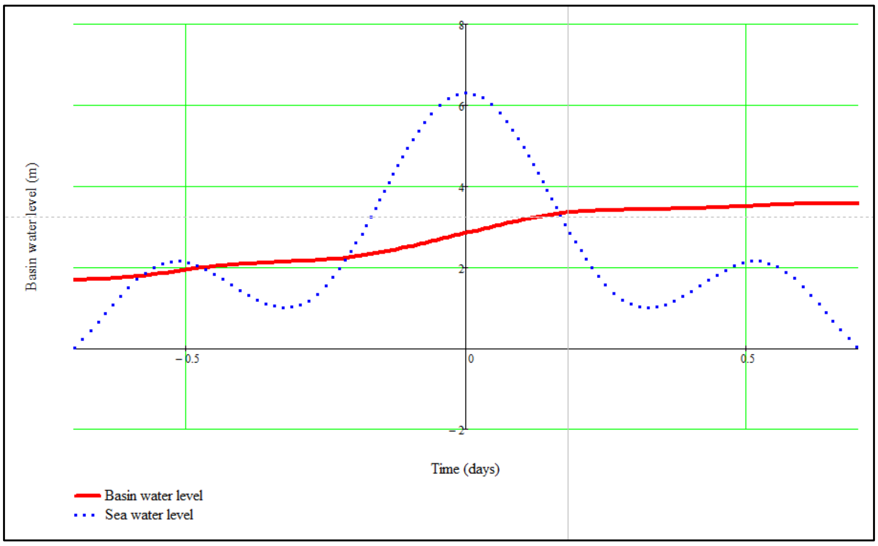
$$h_{in\_fnc}(t, h_{in}, A, A_{IJM}, \Delta, D, h_{s\_peak}) := \left| \begin{array}{l} \Delta t \leftarrow 15 \cdot min \\ \delta \leftarrow 0.01 \cdot m \\ err \leftarrow 100 \cdot m \\ h_{old} \leftarrow 100 \cdot m \\ \text{while } |err| > \delta \\ \quad \left| \begin{array}{l} t_{int} \leftarrow -2 \cdot D \\ h_{int} \leftarrow h_{in} \\ \text{while } t_{int} < t \\ \quad \left| \begin{array}{l} k_1 \leftarrow dh\_dx(t_{int}, h_{in}, A, A_{IJM}, \Delta, D, h_{s\_peak}) \\ k_2 \leftarrow dh\_dx\left(t_{int} + \frac{1}{2} \Delta t, h_{int} + \frac{1}{2} \cdot \Delta t \cdot k_1, A, A_{IJM}, \Delta, D, h_{s\_peak}\right) \\ k_3 \leftarrow dh\_dx\left(t_{int} + \frac{1}{2} \Delta t, h_{int} + \frac{1}{2} \cdot \Delta t \cdot k_2, A, A_{IJM}, \Delta, D, h_{s\_peak}\right) \\ k_4 \leftarrow dh\_dx(t_{int} + \Delta t, h_{int} + \Delta t \cdot k_1, A, A_{IJM}, \Delta, D, h_{s\_peak}) \\ h_{int} \leftarrow h_{int} + \frac{1}{6} \cdot \Delta t \cdot (k_1 + 2 \cdot k_2 + 2 \cdot k_3 + k_4) \\ t_{int} \leftarrow t_{int} + \Delta t \end{array} \right. \\ err \leftarrow (h_{old} - h_{int}) \\ h_{old} \leftarrow h_{int} \\ \Delta t \leftarrow \frac{\Delta t}{2} \end{array} \right. \\ h_{int} \end{array} \right.$$

Sizing a storm surge barrier

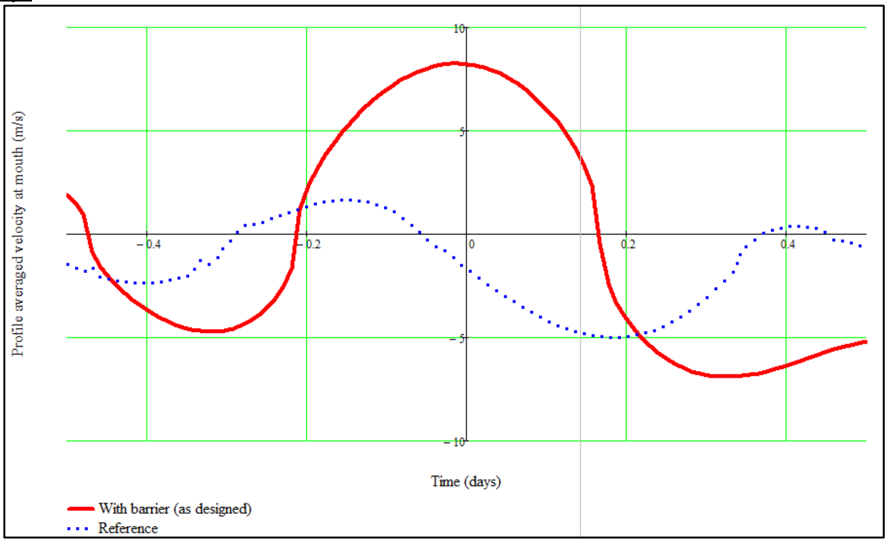
Basin level:

$\gamma := 0.025$ (Opening coefficient)

$A_{barr} := \gamma \cdot 15\,km \cdot 12.5\,m = 4.688 \times 10^{3}\,m^{2}$ (Total opening of barrier)



Current velocity:



K.3 Retaining height of lifting gate

The calculation for limited retaining height of lifting gate in Section 8.2.2 is presented here.

<u>Design storm input</u>

Hw = maximum surge level = 6.3 m

Hs= significant wave height = 3.1 m

Tp= peak wave period = 12 s

g= acceleration of gravity = 9.8 m/s^2

<u>Proposed levee design</u>

Tanα= angle outer slope of the seaside of the levee = 1:1.5 [-]

Rc= retaining height = 8.0 m

γ_β = influence factor for angled wave attack = 1.0 [-]

γ_f = influence factor for roughness elements on slope = 0.7 [-]

γ_b = influence factor for a berm = 1.0 [-]

S_{op}= Wave steepness = 0.015 [-]

ξ= break index = 2.8 [-] >2.0 [-] (non-breaking)

Overtopping discharge output (levee)

Average overtopping discharge (EuroTop Manual, 2007)

$$\frac{q}{\sqrt{gH_{m0}^3}} = \frac{0.067}{\sqrt{tan\alpha}}\gamma_b \xi_{m-1,0} \exp(-4.75\frac{R_C}{\xi_{m-1,0}H_{m0}\gamma_b\gamma_f\gamma_\beta\gamma_v}) \quad \text{(k.6)}$$

With a maximum of:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.2\ exp(-2.6\frac{R_C}{H_{m0}\gamma_f\gamma_\beta}) \quad \text{(k.7)}$$

qn= overtopping for non-breaking waves = 9.66 l/s/m (Max.)

Total overtopping volume = 417,312 [m^3/m]

Length levee = 6.3 [km]

Surface retention area = 1,800 [km^2]

Water level rise inside basin due to dam overtopping = 1.5 m

Overtopping discharge output (gates)

The total overtopping is regarded as a combination of overtopping and overflow. The basic formula for overtopping discharge is given by:

$$q = 0.13\sqrt{gH_S^3}\exp(-3\frac{h_{kr}}{H_S}\frac{1}{\gamma_\beta\gamma_n}) \text{ , given } h_{kr} > 0 \quad \text{(k.8)}$$

With,

q: average overtopping discharge [l/s/m]

g: acceleration of gravity [m/s^2]

H_s: significant wave height in front of gates[m]

H_{kr}: crest height of the structure above the still water line [m]

γ_β: influence factor for angled wave attack [-]

γ_n: influence factor for the shape of the tip of the gate [-]

Overflow is allowed in this condition, which will result a gate height lower than maximum surge level. As stated before, the maximum overtopping and overflow discharge is set at 10 l/s/m. The total discharge over the structure can be calculated according to the following equation:

$$q = 0.6\sqrt{-gh_{kr}^3} + 0.13\sqrt{gH_S^3} \tag{k.9}$$

The first term states the overflow discharge without the influence of waves; the second one describes the maximum overtopping discharge.

Requirement: maximum allowed water level rise in the estuary= 3.5m- 1.5m = 2.0 m

Retaining height of barrier gates	= 5.1 m
Overtopping rate	= 9.49 l/s/m
Barrier length L	= 8.5km +172m= 8.7 km
Surface retention area	= 1,800 [km^2]
Water level rise inside basin due to dam overtopping	= 1.9m

Conclusion: minimum retaining height of the barrier gates without leakage = 5.1m

Appendix L. Swing gate stability check

In this section, checks will be done for the structural and stability design of the gate in different operational phases. The calculations are performed in the Serviceability Limit State (SLS) by considering the material factors. The calculated procedure has been performed to find an optimized design.

L.1 Initial design in reinforcement concrete

This part describes stability check for initial design.

Table L- 1. Initial gate dimensions

Main Dimensions	Parameter	Value	Unit
Length Of Gate	Lc	180	m
Width Of Gate	Wc	26.5	m
Height Of Gate	Hc	21.6	m
Thickness Concrete Slabs	**Parameter**	**Value**	**Unit**
External Wall	Wout	1.5	m
Internal Wall	Win	0.5	m
Top Slab	Wt	1.0	m
Floor Slab	Wf	1.6	m
Compartments	**Parameter**	**Value**	**Unit**
Number in length direction	Nx	15	-
Number in Width Direction	Ny	4	-
Draught and Water Height	**Parameter**	**Value**	**Unit**
Initial Draught Of Gate During Transport	Di	6.83	m
Required Ballasted Water Height	Hb	12.50	m
Desiged Balasted Water Height	Hb,S	14.00	m

Table L- 2. Stability check for floating gate during transport _initial design

Floating Gate during Transport			
Check on shear stength(PER METER)			
occuring shearr force	Fsh,out	238.46	KN
	Fsh,f	547.80	KN
	Fsh,t	225.25	KN
occuring shear stress	tsh,out	238.46	KN/m2
	tsh,f	513.56	KN/m2
	tsh,t	337.88	KN/m2
maximum allowed shear stress	tmax	1680.00	KN/m2
Check on moment capacity(PER METER)			
Local moments	Mw,out	1435.79	KNm
	Mf,middle	4452.00	KNm
	Mf,max	4452.00	KNm
	Mtop	1492.28	KNm
Maximum allowed moment	Mu	48208.57	KNm
Cross sectional area concrete	Ac	154.40	m2
Cross sectional area reinforcement	As	5.40	m2
reinforcement ratio	ps	0.035	-
effective height concrete	d	21.32	m
concrete cover	c	0.30	m
reinforcement diameter		0.032	m

Check on static stability			
mass moment of inertial	Ixx,floor	279144.38	m4
	Iyy,floor	12879000.00	m4
Volume of displaced water	Vdisp	32536.98	m3
Dimensions flooting caisson			
	BK	3.41	m
	BM	8.58	m
	GK	11.26	m
Metacentric height	MG	0.73	m
Minimum metacentric height		0.50	m

Check on dynamic stability(sway)			
	$2\pi W_C$	166.42	
	$2\pi L_C$	1130.40	
Minimum	$gT_{p,reg}^2$	156.80	

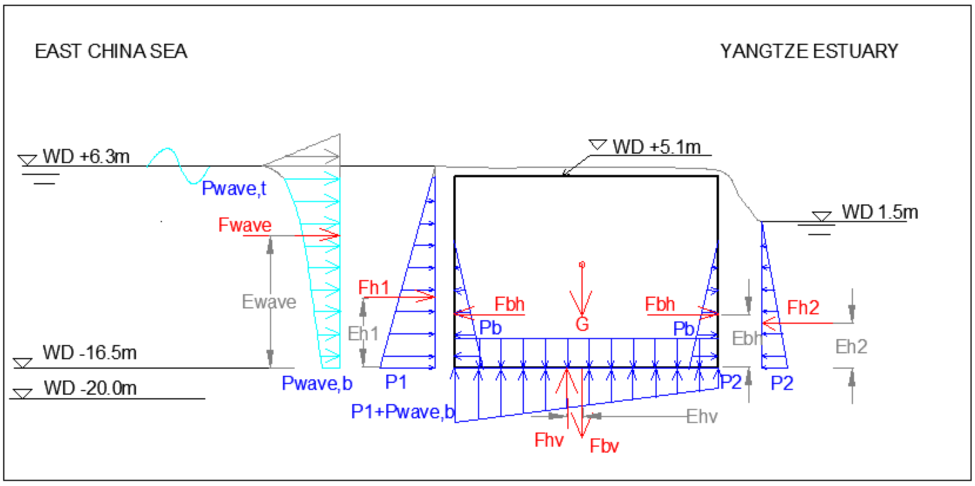
Check on dynamic stability(oscillation)			
Position of neutral axis in the x-direction	Sna,x	9.99	m
Position of neutral axis in the y-direction	Sna,y	9.99	m
Concrete cross section	Ac	154.40	m2
Floor slab area	Af	42.40	m2
Top slab area	At	26.50	m2
Outer wall area	Aw,out	28.50	m2
Inner wall area	Aw,in	9.50	m2
Distance mass center of floor slab and bottom fiber	Sf	0.80	m
Distance mass center of top slab and bottom fiber	St	11.10	m
Distance mass center of outer wall and bottom fiber	Sw,out	11.10	m
Distance mass center of inner wall and bottom fiber	Sw,in	11.10	m
Area of concrete in short side cross-section	Ac,con-x	467.90	m2
Area of concrete in long side cross-section	Ac,con-y	3689.00	m2
Mass moment of inertia in zz-direction along the x-direction	Izz-x	9844.03	m4
Mass moment of inertia in xx-direction along the x-direction	Ixx-x	13633.62	m4
Mass moment of inertia in zz-direction along the y-direction	Izz-y	6927.92	m4
Mass moment of inertia in xx-direction along the y-direction	Ixx-y	1263600.00	m4
Polar moment of inertia radius along x-direction	Sp-x	7.08	m
Polar moment of inertia radius along y-direction	Sp-y	18.56	m
Period of natural oscillation along x-direction	To-x	16.59	s
Period of natural oscillation along y-direction	To-y	43.46	s
Critrial value	Tc	4.00	s

Table L- 3. Static stability check for immersed gate at final location during storm (floating structure) _initial design

Gate Immersed at Final Location during Storm(floating structure)		
Check on static stability		
Design Dc	16.50	m
Weigh Wc	1852.80	KN
Requir Wtot	5247.00	KN
Requir Ww,b	3394.20	KN
Volume per inner caisson	418.00	m3
Requir hb	12.50	m
Mass o Ixx	87720.72	m4
Displac Vdis	78705.00	m3
Dimen BK	8.25	m
BM	1.11	m
GK	4.85	m
Metace MG	4.51	m
Minimum metacentric height	0.50	m

Table L- 4. Force calculation for immersed gate at final location during storm (floating structure) _initial design

Force calculation



Hydrostatic force upsteam			
P1	233.70	KN/m2	
Fh1	2664.18	KN/m	
Eh1	7.60	m	
Hydrostatic force upsteam			
P2	169.13	KN/m2	
Fh2	1395.28	KN/m	
Eh2	5.50	m	
Wave load			
Hi	6.30	m	
Tp	12.00	s	
d	20.00	m	
L	229.30	m	
k	0.03	-	
Pwave,t	64.58	kN/m3	
Pwave,b	56.22	kN/m3	
Fwave	1377.11	kN/m	
Pressure from ballasted water			
Hballasted,s	14.00	m	(~12.5+1.5)
Pb	143.56	kN/m3	
Fbh	1005.35	kN/m	

Table L- 5. Structural stability check for elements of immersed gate at final location during storm (floating structure) _initial design

Check on structural stability			
Shear stress check wall			
dw	1.22	m	
Crd,c	0.12	-	
k	1.41	-	
percentage reinforcement	0.02	-	
fck	67.00	Mpa	
bw	1.00	m	
cot theta	1.62	-	theta=45
z	1.09	m	
s	0.30	m	
fywd	435.00	Mpa	
Asw	695.61	mm2/m	
Assumed steel diameter	18.00	mm2/m	
Asw,o	254.34	mm2/m	
Nsw	2.73	-	
Moment strength check wall			
xw	76.10	-	
dw	1.22	m	
percentage	0.01	-	
As	12160.00	mm2/m	
Mu	7048.79	kNm/m	
Md	6283.00	kNm/m	
Assumed steel diameter	32.00	mm	
Abar	803.84	mm2	
Nbar	15.13	-	

Shear stress check top slab		
dt	0.72	m
Crd,c	0.12	
k	1.53	
percentage1	0.02	
fck	67.00	Mpa
bt	1.00	m
cot theta	1.62	
z	0.64	m
s	0.30	m
fywd	435.00	Mpa
Asw	223.00	mm2/m
steel	18.00	mm
Asw,o	254.34	mm2/m
Nsw	0.87	
Moment strength check top slab		
xt	34.76	
dt	0.72	m
percentage	0.006	
As	4296.00	mm2/m
Mu	1496.00	kNm/m
Md	995.00	kNm/m
Steel assumed	32.00	mm
Abar	803.84	mm2
Nbar	5.34	

Shear stress check floor slab		
df	1.316	m
Crd,c	0.12	
k	1.39	
percentage	0.02	
fck	67	Mpa
bf	1	m
cot theta	1.62	
z	1.18	m
s	0.30	m
fywd	435.00	Mpa
Asw	412.70	mm2/m
steel	18.00	mm
Asw,o	254.34	mm2/m
Nsw	1.62	
Moment strength check floor slab		
xf	58.82	
df	1.31	m
percentage	0.010	
As	13160.00	mm2/m
Mu	8271.00	kNm/m
Md	5688.00	kNm/m
Steel assumed	32.00	mm
Abar	804.00	mm2
Nbar	16.37	

Table L- 6. Stability check for immersed gate at final location during storm (resting on abutments) _initial design

Gate Immersed at Final Location during Storm(resting on abutments)			
Force calculation			
q	120.80	kN/m2	
L	180.00	m	
Mmax	489236.92	kNm/m	
Vmax	10871.93	kN/m	
t	21.60	m	
d	21.28	m	
Moment check			
Asw,req	63852.00	mm2	
Bars assumed	32.00	mm	
Ab	803.84	mm2	
Nb	79.43		
hbb	80.00	mm	Heart to heart of bars
rows	9.93		Number of rows of bars per meter
percentage	0.003		
Mu	582917.47	kNm/m	
Mmax	489236.92	kNm/m	
Shear reinforcement			
cot theta	1.62		
z	1.09	m	
s	0.30	m	
Asw	4229.68	mm2	
Stirrups assumed as			
steel	18.00	mm	
Asw,o	254.34	mm2	
Nsw	16.63		

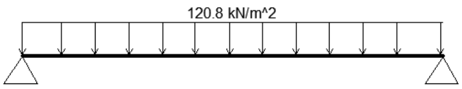
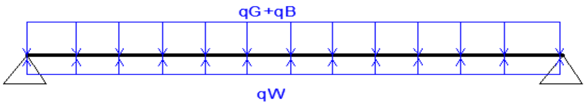


Table L- 7. Stability check for immersed gate at final location during storm (resting on foundation) _initial design

Gate Immersed at Final Location during Storm(resting on foundation)

Force calculation

qg	1852.80	KN /m
qb	18.86	KN /m
qs	59.45	KN /m
qw	1691.25	KN /m
qtot	239.86	KN /m
Vmax	21587.10	KN
Mmax	971419.50	KNm



Moment check

As,req	116.58	mm2	
Bars assumed	32.00	mm	
Ab	803.84	mm2	
Nb	26.11		
hbb	80.00	mm	Heart to heart of bars
rows	3.26		Number of rows of bars per meter
percentage	0.0055		
Md	1051824.59	KNm	
Mmax	643072.00	KNm	

Shear reinforcement

cot theta	1.62	
z	1.18	m
s	0.30	m
Asw	7760.20	mm2
Stirrups assumed as		
steel	18.00	mm
Asw,o	254.34	mm2
Nsw	30.51	

Table L- 8. Static stability for immersed gate at final location after storm (negative head) _initial design

Gate Immersed at Final Location during Storm(floating structure)

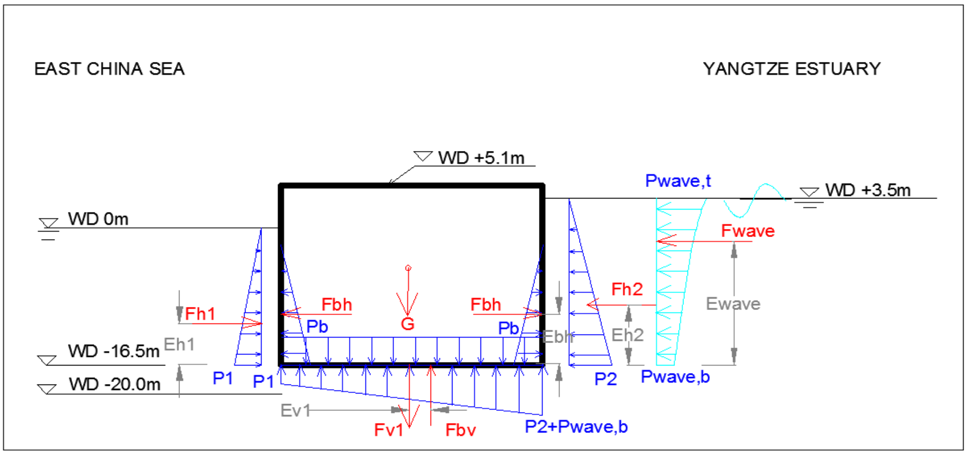
Check on static stability

Design Dc	16.50	m
Weigh Wc	1852.80	KN
Requir Wtot	5247.00	KN
Requir Ww,b	3394.20	KN
Volume per inner caisson	418.00	m3
Requir hb	12.50	m
Mass o Ixx	87720.72	m4
Displac Vdis	78705.00	m3
Dimen BK	8.25	m
BM	1.11	m
GK	4.85	m
Metace MG	4.51	m
Minimum metacentric height	0.50	m

Table L- 9. Force calculation for immersed gate at final location after storm (negative head) _initial design

Gate Immersed at Final Location after Storm (negtive head)

Force calculation



Hydrostatic force upsteam

P1	169.13	KN/m2
Fh1	1395.28	KN/m
Eh1	5.50	m

Hydrostatic force upsteam

P2	240.88	KN/m2
Fh2	2830.28	KN/m
Eh2	7.83	m

Wave load

Hi	3.50	m
Tp	4.00	s
d	20.00	m
L	25.48	m
k	0.25	-
Pwave,t	35.88	kN/m3
Pwave,b	0.72	kN/m3
Fwave	301.94	kN/m

Pressure from ballasted water

Hballasted	14.00	m
Pb	143.56	kN/m3
Fbh (Fbv)	1005.35	kN/m

Table L- 10. Structural stability for elements of immersed gate at final location after storm (negative head) _initial design

Check on structural stability		
Shear stress check wall		
dw	1.22	m
Crd,c	0.12	
k	1.41	
percentage	0.02	
fck	67.00	Mpa
bw	1.00	m
Vrd,c	197.31	kN
cot theta	1.62	
z	1.09	m
s	0.30	m
fywd	435.00	Mpa
Asw	420.00	mm2/m
steel	18.00	mm
Asw,o	254.34	mm2/m
Nsw	1.64	
Moment strength check wall		
xw	43.64	
dw	1.22	m
percentage	0.006	
As	7296.00	mm2/m
Mu	4311.95	kNm/m
Md	3603.00	kNm/m
Steel assum	32.00	mm
Abar	803.84	mm2
Nbar	9.08	

Shear stress check floor slab		
df	1.32	m
Crd,c	0.12	
k	1.39	
percentage	0.02	
fck	67.00	Mpa
bf	1.00	m
cot theta	1.62	
z	1.18	m
s	0.30	m
fywd	435.00	Mpa
Asw	370.00	mm2/m
steel	18.00	mm
Asw,o	250.00	mm2/m
Nsw	1.44	
Moment strength check floor slab		
xf	42.24	
df	1.32	m
percentage	0.005	
As	6580.00	mm2/m
Mu	4232.65	kNm/m
Md	4084.00	kNm/m
Steel assum	32.00	mm
Abar	803.84	mm2
Nbar	8.12	

Shear stress check top slab		
dt	0.72	m
Crd,c	0.12	
k	1.53	
percentage	0.02	
fck	67.00	Mpa
bt	1.00	m
cot theta	1.62	
z	0.64	m
s	0.30	m
fywd	435.00	Mpa
Asw	220.00	mm2/m
steel	18.00	mm
Asw,o	254.34	mm2/m
Nsw	0.87	
Moment strength check top slab		
xt	34.76	
dt	0.72	m
percentage	0.005	
As	3080.00	mm2/m
Mu	1252.93	kNm/m
Md	995.00	kNm/m
Steel assum	32.00	mm
Abar	803.84	mm2
Nbar	4.45	

Table L- 11. Summary of stability check of initial design

Aspects	Design Parameter	Value	Requirment	
Floating Gate During Transport				
Structural Check	Bending Moment Wall	33.5	Should Be Larger Than 1	Y
	Bending Moment Floor Slab	10.8	Should Be Larger Than 1	Y
	Bending Moment Top Slab	32.3	Should Be Larger Than 1	Y
	Shear Stress Wall	7.04	Should Be Larger Than 1	Y
	Shear Stress Floor Slab	3.27	Should Be Larger Than 1	Y
	Shear Stress Top Slab	4.97	Should Be Larger Than 1	Y
Static Stability Check	Minimum Metacentric Height	0.73m	Should Be Larger Than 0.5m	Y
Dynamic Stabiity Check	Sway	7.01	Should Be Large Than 1	Y
	Period Of Natural Oscillation To-X	4.14	Should Be Large Than 2	Y
	Period Of Natural Oscillation To-Y	10.8	Should Be Large Than 2	Y
Gate Immersed At Final Location During Storm (Floating Stucture)				
Structural Check	Bending Moment Wall	1.21	Should Be Larger Than 1	Y
	Bending Moment Floor Slab	1.45	Should Be Larger Than 1	Y
	Bending Moment Top Slab	1.5	Should Be Larger Than 1	Y
	Shear Stress Wall	1.01	Should Be Larger Than 1	Y
	Shear Stress Floor Slab	1.56	Should Be Larger Than 1	Y
	Shear Stress Top Slab	4.97	Should Be Larger Than 1	Y
Static Stability Check	Minimum Metacentric Height	4.5m	Should Be Larger Than 0.5m	Y
Gate Immersed At Final Location During Storm (Resting On Abutments)				
Structural Check	Bending Moment External Wall	1.19	Should Be Larger Than 1	Y
	Shear Stress External Wall	1.21	Should Be Larger Than 1	Y
Gate Immersed At Final Location During Storm (Resting On Foundation)				
Structural Check	Bending Moment Floor Slab	1.63	Should Be Larger Than 1	Y
	Shear Stress Floor Slab	1.11	Should Be Larger Than 1	Y
Gate Immersed At Final Location After Storm (Negtive Head)				
Structural Check	Bending Moment Wall	1.19	Should Be Larger Than 1	Y
	Bending Moment Floor Slab	1.04	Should Be Larger Than 1	Y
	Bending Moment Top Slab	1.26	Should Be Larger Than 1	Y
	Shear Stress Wall	1.56	Should Be Larger Than 1	Y
	Shear Stress Floor Slab	1.75	Should Be Larger Than 1	Y
	Shear Stress Top Slab	4.97	Should Be Larger Than 1	Y

L.2 Optimized design in reinforcement concrete

Table L- 12. Optimized gate dimensions

Main Dimensions	Parameter	Value	Unit
Length Of Gate	Lc	180	m
Width Of Gate	Wc	26.5	m
Height Of Gate	Hc	21.6	m
Thickness Concrete Slabs	**Parameter**	**Value**	**Unit**
External Wall	Wout	1.5	m
Internal Wall	Win	0.4	m
Top Slab	Wt	0.9	m
Floor Slab	Wf	1.5	m
Compartments	**Parameter**	**Value**	**Unit**
Number in length direction	Nx	15	-
Number in Width Direction	Ny	4	-
Draught and Water Height	**Parameter**	**Value**	**Unit**
Initial Draught Of Gate During Transport	Di	6.37	m
Required Ballasted Water Height	Hb	12.94	m
Desiged Balasted Water Height	Hb,S	14.45	m

Table L- 13. Stability check for floating gate during transport _optimized design

Floating Gate during Transport			
Check on shear stength(PER METER)			
occuring shearr force	Fsh,out	208.11	KN
	Fsh,f	531.90	KN
	Fsh,t	202.73	KN
occuring shear stress	tsh,out	208.11	KN/m2
	tsh,f	531.90	KN/m2
	tsh,t	337.88	KN/m2
maximum allowed shear stress	tmax	1680.00	KN/m2
Check on moment capacity(PER METER)			
Local moments	Mw,out	1170.05	KNm
	Mf,middle	4153.48	KNm
	Mf,max	4153.48	KNm
	Mtop	1343.05	KNm
Maximum allowed moment	Mu	45036.30	KNm
Cross sectional area concrete	Ac	144.24	m2
Cross sectional area reinforcement	As	5.05	m2
reinforcement ratio	ps	0.04	-
effective height concrete	d	21.32	m
concrete cover	c	0.30	m
reinforcement diameter		0.03	m

Check on static stability			
mass moment of inertial	Ixx,floor	279144.38	m4
	Iyy,floor	12879000.00	m4
Volume of displaced water	Vdisp	30395.94	m3
Dimensions flooting caisson			
	BK	3.19	m
	BM	9.18	m
	GK	11.25	m
Metacentric height	MG	1.12	m
Minimum metacentric height		0.50	m

Check on dynamic stability(sway)			
	$2\pi W_C$	166.42	
	$2\pi L_C$	1130.40	
Minimum	$gT_{p,reg}^2$	156.80	

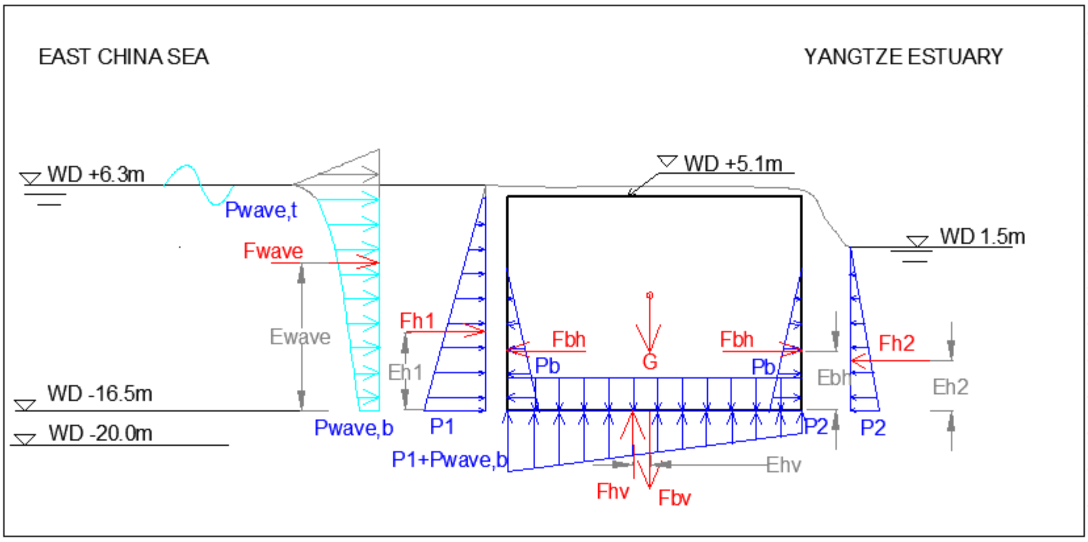
Check on dynamic stability(oscillation)			
Position of neutral axis in the x-direction	Sna,x	9.91	m
Position of neutral axis in the y-direction	Sna,y	9.91	m
Concrete cross section	Ac	144.24	m2
Floor slab area	Af	39.75	m2
Top slab area	At	23.85	m2
Outer wall area	Aw,out	28.80	m2
Inner wall area	Aw,in	7.68	m2
Distance mass center of floor slab and bottom fiber	Sf	0.75	m
Distance mass center of top slab and bottom fiber	St	11.10	m
Distance mass center of outer wall and bottom fiber	Sw,out	11.10	m
Distance mass center of inner wall and bottom fiber	Sw,in	11.10	m
Area of concrete in short side cross-section	Ac,con-x	465.36	m2
Area of concrete in long side cross-section	Ac,con-y	3688.80	m2
Mass moment of inertia in zz-direction along the x-direction	Izz-x	9140.45	m4
Mass moment of inertia in xx-direction along the x-direction	Ixx-x	13281.39	m4
Mass moment of inertia in zz-direction along the y-direction	Izz-y	6409.86	m4
Mass moment of inertia in xx-direction along the y-direction	Ixx-y	1166400.00	m4
Polar moment of inertia radius along x-direction	Sp-x	6.94	m
Polar moment of inertia radius along y-direction	Sp-y	17.83	m
Period of natural oscillation along x-direction	To-x	13.16	s
Period of natural oscillation along y-direction	To-y	33.81	s
Critrial value	Tc	4.00	s

Table L- 14. Static stability check for immersed gate at final location during storm (floating structure) _optimized design

Gate Immersed at Final Location during Storm(floating structure)			
Check on static stability			
Design	Dc	16.50	m
Weigh	Wc	1730.88	KN
Requir	Wtot	5247.00	KN
Requir	Ww,b	3516.12	KN
Volume per inner caisson		428.16	m3
Requir	hb	12.94	m
Mass o	Ixx	87720.72	m4
Displac	Vdis	78705.00	m3
Dimen	BK	8.25	m
	BM	1.11	m
	GK	4.53	m
Metace	MG	4.84	m
Minimum metacentric height		0.50	m

Table L- 15. Force calculation for immersed gate at final location during storm (floating structure) _optimized design

Force calculation



Hydrostatic force upsteam			
P1	233.70	KN/m2	
Fh1	2664.18	KN/m	
Eh1	7.60	m	
Hydrostatic force upsteam			
P2	169.13	KN/m2	
Fh2	1395.28	KN/m	
Eh2	5.50	m	
Wave load			
Hi	6.30	m	
Tp	12.00	s	
d	20.00	m	
L	229.30	m	
k	0.03	-	
Pwave,t	64.58	kN/m3	
Pwave,b	56.22	kN/m3	
Fwave	1377.11	kN/m	
Pressure from ballasted water			
Hballasted,s	14.45	m	(~12.94+1.5)
Pb	148.16	kN/m3	
Fbh	1070.82	kN/m	

Table L- 16. Structural stability check for elements of immersed gate at final location during storm (floating structure) _optimized design

Check on structural stability			
Shear stress check wall			
dw	1.22	m	
Crd,c	0.12	-	
k	1.41	-	
percentage reinforcement	0.02	-	
fck	67.00	Mpa	
bw	1.00	m	
Vrd,c	327.18	kN	
Vmax.wall	1788.00	kN	sea side
Vmax,wall	299.00	kN	estuary side
cot theta	1.62	-	theta=45
z	1.09	m	
s	0.30	m	
fywd	435.00	Mpa	
Asw	695.61	mm2/m	
Assumed steel diameter	20.00	mm2/m	
Asw,o	254.34	mm2/m	
Nsw	2.73	-	
Moment strength check wall			
xw	76.10	-	
dw	1.22	m	
percentage	0.01	-	
As	12160.00	mm2/m	
Mu	7048.79	kNm/m	
Md	6283.00	kNm/m	
Assumed steel diameter	30.00	mm	
Abar	803.84	mm2	
Nbar	15.13	-	

Shear stress check floor slab		
df	1.216	m
Crd,c	0.12	
k	1.405553553	
percentage	0.02	
fck	67	Mpa
bf	1	m
cot theta	1.62	
z	1.09	m
s	0.30	m
fywd	435.00	Mpa
Asw	446.60	mm2/m
steel	18.00	mm
Asw,o	254.34	mm2/m
Nsw	1.75	
Moment strength check floor slab		
xf	68.90	
df	1.21	m
percentage	0.010	
As	12160.00	mm2/m
Mu	7062.00	kNm/m
Md	5688.00	kNm/m
Steel assumed	30.00	mm
Abar	803.84	mm2
Nbar	15.12	

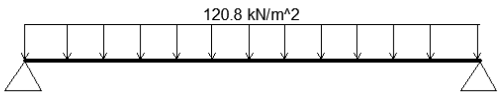
Shear stress check top slab		
dt	0.62	m
Crd,c	0.12	
k	1.57	
percentage1	0.02	
fck	67.00	Mpa
bt	1.00	m
cot theta	1.62	
z	0.55	m
s	0.30	m
fywd	435.00	Mpa
Asw	288.00	mm2/m
steel	18.00	mm
Asw,o	254.34	mm2/m
Nsw	1.13	
Moment strength check top slab		
xt	46.96	
dt	0.62	m
percentage	0.006	
As	3696.00	mm2/m
Mu	1107.77	kNm/m
Md	995.00	kNm/m
Steel assumed	32.00	mm
Abar	803.84	mm2
Nbar	4.60	

Table L- 17. Stability check for immersed gate at final location during storm (resting on abutments) _optimized design

Gate Immersed at Final Location during Storm(resting on abutments)

Force calculation

q	120.80	kN/m2	
L	180.00	m	
Mmax	489236.92	kNm/m	
Vmax	10871.93	kN/m	
t	21.60	m	
d	21.28	m	



Moment check

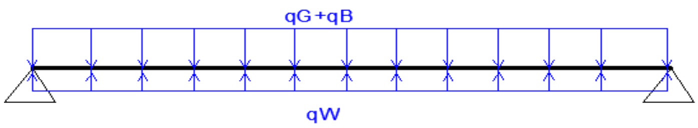
Asw,req	63852.00	mm2	
Bars assumed	32.00	mm	
Ab	803.84	mm2	
Nb	79.43		
hbb	80.00	mm	Heart to heart of bars
rows	9.93		Number of rows of bars per meter
percentage	0.003		
Mu	582917.47	kNm/m	
Mmax	489236.92	kNm/m	

Shear reinforcement

cot theta	1.62	
z	1.09	m
s	0.30	m
Asw	4229.68	mm2
Stirrups assumed as		
steel	18.00	mm
Asw,o	254.34	mm2
Nsw	16.63	

Table L- 18. Stability check for immersed gate at final location during storm (resting on foundation) _optimized design

Gate Immersed at Final Location during Storm(resting on foundation)		
Force calculation		
qg	1730.88	KN /m
qb	19.53	KN /m
qs	59.45	KN /m
qw	1691.25	KN /m
qtot	118.61	KN /m
Vmax	10675.26	KN
Mmax	643072.00	KNm



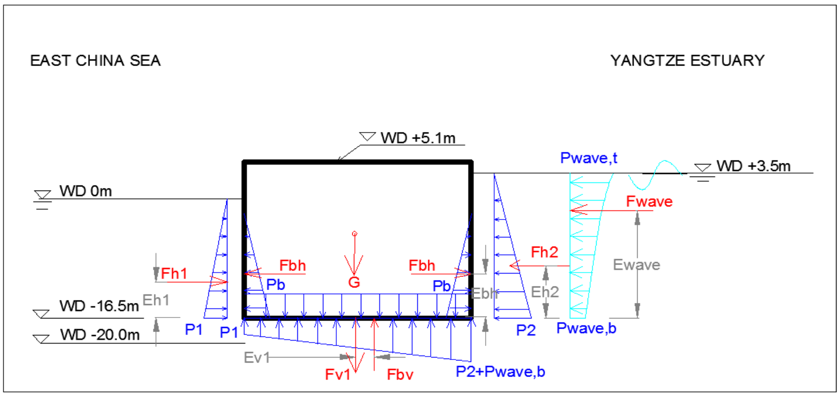
Moment check			
As,req	57.65	mm2	
Bars assumed	32.00	mm	
Ab	803.84	mm2	
Nb	12.91		
hbb	80.00	mm	Heart to heart of bars
rows	1.61		Number of rows of bars per meter
percentage	0.0027		
Md	527030.43	KNm	
Mmax	643072.00	KNm	

Shear reinforcement		
cot theta	1.62	
z	1.09	m
s	0.30	m
Asw	4153.17	mm2
Stirrups assumed as		
steel	18.00	mm
Asw,o	254.34	mm2
Nsw	16.33	

Table L- 19. Force calculation for immersed gate at final location after storm (negative head) _optimized design

Gate Immersed at Final Location after Storm (negtive head)

Force calculation



Hydrostatic force upsteam			
	P1	169.13	KN/m2
	Fh1	1395.28	KN/m
	Eh1	5.50	m
Hydrostatic force upsteam			
	P2	240.88	KN/m2
	Fh2	2830.28	KN/m
	Eh2	7.83	m
Wave load			
	Hi	3.50	m
	Tp	4.00	s
	d	20.00	m
	L	25.48	m
	k	0.25	-
	Pwave,t	35.88	kN/m3
	Pwave,b	0.72	kN/m3
	Fwave	301.94	kN/m
Pressure from ballasted water			
	Hballasted	14.45	m
	Pb	148.16	kN/m3
	Fbh (Fbv)	1070.82	kN/m

Table L- 20. Stability check for elements of immersed gate at final location after storm (negative head) _optimized design

Check on structural stability								
Shear stress check wall			***Shear stress check floor slab***			***Shear stress check top slab***		
dw	1.22	m	df	1.22	m	dt	0.62	m
Crd,c	0.12		Crd,c	0.12		Crd,c	0.12	
k	1.41		k	1.41		k	1.57	
percentage	0.02		percentage	0.02		percentage	0.02	
fck	67.00	Mpa	fck	67.00	Mpa	fck	67.00	Mpa
bw	1.00	m	bf	1.00	m	bt	1.00	m
Vrd,c	197.31	kN	cot theta	1.62		cot theta	1.62	
cot theta	1.62		z	1.09	m	z	0.55	m
z	1.09	m	s	0.30	m	s	0.30	m
s	0.30	m	fywd	435.00	Mpa	fywd	435.00	Mpa
fywd	435.00	Mpa	Asw	397.22	mm2/m	Asw	288.00	mm2/m
Asw	418.61	mm2/m	steel	18.00	mm	steel	18.00	mm
steel	18.00	mm	Asw,o	254.34	mm2/m	Asw,o	254.34	mm2/m
Asw,o	254.34	mm2/m	Nsw	1.56		Nsw	1.13	
Nsw	1.65							
Moment strength check wall			***Moment strength check floor slab***			***Moment strength check top slab***		
xw	43.64		xf	49.47		xt	46.96	
dw	1.22	m	df	1.22	m	dt	0.62	m
percentage	0.01		percentage	0.005		percentage	0.005	
As	7296.00	mm2/m	As	6080.00	mm2/m	As	3080.00	mm2/m
Mu	4311.95	kNm/m	Mu	3613.83	kNm/m	Mu	927.39	kNm/m
Md	3603.00	kNm/m	Md	4084.00	kNm/m	Md	920.00	kNm/m
Steel assur	32.00	mm	Steel assur	30.00	mm	Steel assur	32.00	mm
Abar	803.84	mm2	Abar	803.84	mm2	Abar	803.84	mm2
Nbar	9.08		Nbar	7.56		Nbar	3.83	

Table L- 21. Summary of stability check of optimized design

Aspects	Design Parameter	Value	Requirment	
		Floating Gate During Transport		
Structural Check	Bending Moment Wall	38.49	Should Be Larger Than 1	Y
	Bending Moment Floor Slab	10.84	Should Be Larger Than 1	Y
	Bending Moment Top Slab	33.53	Should Be Larger Than 1	Y
	Shear Stress Wall	8.07	Should Be Larger Than 1	Y
	Shear Stress Floor Slab	3.16	Should Be Larger Than 1	Y
	Shear Stress Top Slab	4.97	Should Be Larger Than 1	Y
Static Stability Check	Minimum Metacentric Height	1.12	Should Be Larger Than 0.5m	Y
Dynamic Stabiity Check	Sway (Dimensions Of Floating Structure)	2.24	Should Be Large Than 1	Y
	Period Of Natural Oscillation To-X	13.16	Should Be Large Than 2	Y
	Period Of Natural Oscillation To-Y	33.81	Should Be Large Than 2	Y
		Gate Immersed At Final Location During Storm (Floating Stucture)		
Structural Check	Bending Moment Wall	1.12	Should Be Larger Than 1	Y
	Bending Moment Floor Slab	1.24	Should Be Larger Than 1	Y
	Bending Moment Top Slab	1.11	Should Be Larger Than 1	Y
	Shear Stress Wall	1.01	Should Be Larger Than 1	Y
	Shear Stress Floor Slab	1.46	Should Be Larger Than 1	Y
	Shear Stress Top Slab	4.48	Should Be Larger Than 1	Y
Static Stability Check	Minimum Metacentric Height	4.84	Should Be Larger Than 0.5m	Y
		Gate Immersed At Final Location During Storm (Resting On Abutments)		
Structural Check	Bending Moment External Wall	1.19	Should Be Larger Than 1	Y
	Shear Stress External Wall	1.02	Should Be Larger Than 1	Y
		Gate Immersed At Final Location During Storm (Resting On Foundation)		
Structural Check	Bending Moment Floor Slab	1.12	Should Be Larger Than 1	Y
	Shear Stress Floor Slab	1.01	Should Be Larger Than 1	Y
		Gate Immersed At Final Location After Storm (Negtive Head)		
Structural Check	Bending Moment Wall	1.20	Should Be Larger Than 1	Y
	Bending Moment Floor Slab	1.02	Should Be Larger Than 1	Y
	Bending Moment Top Slab	1.21	Should Be Larger Than 1	Y
	Shear Stress Wall	1.56	Should Be Larger Than 1	Y
	Shear Stress Floor Slab	1.65	Should Be Larger Than 1	Y
	Shear Stress Top Slab	4.48	Should Be Larger Than 1	Y

L.3 Optimized design in prestressed concrete

Table L- 22. Final design barge gate in prestressed concrete

Main Dimensions	Parameter	Value	Unit
Length Of Gate	Lc	180	m
Width Of Gate	Wc	26.5	m
Height Of Gate	Hc	21.6	m
Thickness Concrete Slabs	**Parameter**	**Value**	**Unit**
External Wall	Wout	1.2	m
Internal Wall	Win	0.3	m
Top Slab	Wt	1.0	m
Floor Slab	Wf	1.5	m
Compartments	**Parameter**	**Value**	**Unit**
Number in Length Direction	Nx	15	-
Number in Width Direction	Ny	4	-
Draught and Water Height	**Parameter**	**Value**	**Unit**
Initial Draught Of Gate During Transport	Di	5.71	m
Required Ballasted Water Height	Hb	13.60	m
Desiged Balasted Water Height	Hb,S	14.00	m

Table L- 23. Selected steel characteristics

steel type **Y1860S7**	fpk	1860	MPa
	fpk/rs	1691	MPa
	fp0.1k	1674	MPa
	op,max	1590	MPa
	opm0	1395	MPa
	fpd	1522	MPa
	Ep	195000	N/mm2

Floating caisson during transport

Wall

ep	1	m	
Wc	92.45	m3	
qp	92	kN/m	
Ac	25.8	m2	
qg	645	kN/m	
R	80	m	
Pm	7360	kN	
σ_{m0}	1395	N/mm2	initial prestressing stress
Ap,b	1050	mm2	strand diameter 15.7mm
Np	5		

Floor slab

ep	1.1	m	
Wc	9.9375	m3	
qp	61	kN/m	
Ac	39.75	m2	
qg	993.75	kN/m	
Pm	60618.75	kN	
σ_{m0}	1395	N/mm2	initial prestressing stress
Ap,b	1050	mm2	strand diameter 15.7mm
Np	41		

Top slab

ep	0.8	m	
Wc	4.416667	m3	
qp	10.2	kN/m	
Ac	26.5	m2	
qg	662.5	kN/m	
Pm	6757.5	kN	
σ_{m0}	1395	N/mm2	initial prestressing stress
Ap,b	1050	mm2	strand diameter 15.7mm
Np	5		

Gate immersed at finallocation during storm				
Wall				
	ep	1	m	
	Wc	92.45	m3	
	qp	212	kN/m	
	Ac	25.8	m2	
	qg	645	kN/m	
	R	80	m	
	Pm	16960	kN	
	σ_{m0}	1395	N/mm2	initial prestressing stress
	Ap,b	1050	mm2	strand diameter 15.7mm
	Np	12		
Floor slab				
	ep	1.1	m	
	Wc	9.9375	m3	
	qp	153	kN/m	
	Ac	39.75	m2	
	qg	993.75	kN/m	
	Pm	152043.75	kN	
	σ_{m0}	1395	N/mm2	initial prestressing stress
	Ap,b	1050	mm2	strand diameter 15.7mm
	Np	104		
Top slab				
	ep	0.8	m	
	Wc	4.416666667	m3	
	qp	10.2	kN/m	
	Ac	26.5	m2	
	qg	662.5	kN/m	
	Pm	6757.5	kN	
	σ_{m0}	1395	N/mm2	initial prestressing stress
	Ap,b	980	mm2	strand diameter 15.3mm
	Np	5		

Gate resting on abutments			
ep	13	m	
qp	2887.9	kN/m	
R	550	m	
Pm	794178	kN	consideing top and botton slabs for prestressing
σ_{m0}	1395	N/mm2	initial prestressing stress
Ap,b	1050	mm2	strand diameter 15.7mm
Np	542		

Gate resting on foundation			
ep	9.5	m	
qp	2887.9	kN/m	
R	400	m	
Pm	231033.6	kN	consideing all walls for prestressing
σ_{m0}	1395	N/mm2	initial prestressing stress
Ap,b	1050	mm2	strand diameter 15.7mm
Np	158		

Appendix M. Abutment design of barge gate

M.1 Force calculation

M.1.1 Hydraulic force during storm

Hydraulic force during storm				
Distributed load from seaside on gate	q	120.8	kN/m2	
	q	2609.3	kN/m	(~120.8*21.6)
Support reaction force	Fa	234835.2	kN	
Moment to the abutment bottom	M1	2700604.8	kNm	

M.1.2 Berthing force during berthing

number of fenders in vertical direction	Nf	2	-	
*assuming 2 fenders in the height of 7m of the abutments				
stiffness of structure	k	920	kN/m	
design velocity of berthing gate	Vg	0.15	m/s	
hydrodynamic coefficient	**CH**	**1.2**	**-**	
density of sea water		1025	kg/m3	
length of gate	L	180	m	
draught of gate	D	7	m	~6.82m
mass of additional water	mw	7887	kg	
block coefficient	CB	0.8	-	
width of gate	B	26.5	m	
mass of gate	mg	56244	kg	
eccentricity coefficient	**CE**	**0.377**	**-**	
radious of gyration	k	47.16	-	
angle between r and gate velocity	γ	45	degree	
	cos(γ)	0.53	-	
radius between mass center and collision point	r	120.2		
softness coefficient	**CS**	**0.9**	**-**	
configuration coefficient	**CC**	**1**	**-**	
total berthing kinetic energy	Ekin,t	515.0	kNm	
berthing kinetic energy of each fender	Ekin,i	257.5	kNm	
total berthing force	Fbet	973.5	kN	
berthing force of each fender	Fbet,i	486.7	kN	

Berthing force during closure			
Berthing force of each fender	Fbet,	486.7	kN
Upper fender to the bottom	e1	16.9	m
Lower fender to the bottom	e2	19.3	m
Moment to the abutment bottom	M2	17619.8	kNm

M.2 Structural design

Normal concrete				
Abutment height	Hb	23.0	m	
Abutment length	Lb	8.0	m	
Abutment width	Wb	6.0	m	
density of concrete		17.0	kN/m3	
Effective depth	d	7.8	m	
	fywd	435.0	N/mm2	steel B500B
Required steel	Areq	884371.4	mm2	
Bars assumed as		40.0	mm	
	Abar	1256.0	mm2	
Required number of bars	Nbar	704.1		
	ht	80.0	mm	
	row	8.8		
	As	60372000.0	mm2	
Weight of bars	wb	55766.1	kg	
Weight of concrete	wc	1876800.0	kg	
Total weight of one abutment	wtot	1932566.1	kg	
		1932.6	ton	
Prestressed concrete				
Abutment height	Hb	23.0	m	
Abutment length	Lb	6.0	m	
Abutment width	Wb	5.0	m	
density of concrete		17.0	kN/m3	
Required prestressing force	Pm	771601.4	kN	
Initial stress		1395.0	N/mm2	tendon Y1860
	Ap	6000.0	mm2	strand 30 18
Required	Ar	553119.3	mm2	
Required tendons	Nr	92.2		
Weight of concrete	wc	1173000.0	kg	
weight of tendons	wter	33021.2	kg	
Total weight of one abutment	wtot	1206021.2	kg	
		1206.0	ton	

Appendix N. Foundation Design

This part correspond to Section 9.10 in the main the report. The calculations on the foundation design of swing gate is presented.

N.1 Untreated subsoil

Koppejan method

Consolidation duration:

$$t_{99\%} = \frac{1.78w^2}{c_v}$$

$$c_v = \frac{v_v E_s}{\rho_{sw}}$$

Layer	Depth	Thichness	Class	Undrained shear stress	Internal friction angle	Vertical permeability	Young's modulus	Consolidation duration			Vertical consolidation constant
	WD	D			ϕ	P	Es		t		Cv
	(-m)	(m)		(kPa)		(m/s)	(kN/m2)	(s)	(day)	(yr)	(m2/s)
K1	0~5	5	soft clay	14	25	1.E-09	3000	1.E+07	172	0.47	3.0E-06
K2	5~13	8	silty clay	34	30	5.E-09	2500	9.E+06	105	0.29	1.3E-05
K3	13~31	18	silt	25	34	5.E-09	11000	1.E+07	121	0.33	5.5E-05
K4	31~45	14	firm clay	28	35	1.E-09	3000	1.E+08	1346	3.69	3.0E-06
K5	45~65	20	silty sand	48	35	8.E-10	14000	6.E+07	736	2.02	1.1E-05
K6	below 65		very dense sand								

From the result, it appears the layer K5 is the governing one. It will take 1,346 days to fully consolidate. One to be noted is that the Koppejan method is only suitable for one time compression, but soil recompression in the later design stage might be relevant.

N.2 Preloading the subsoil

Unit time and cost rates

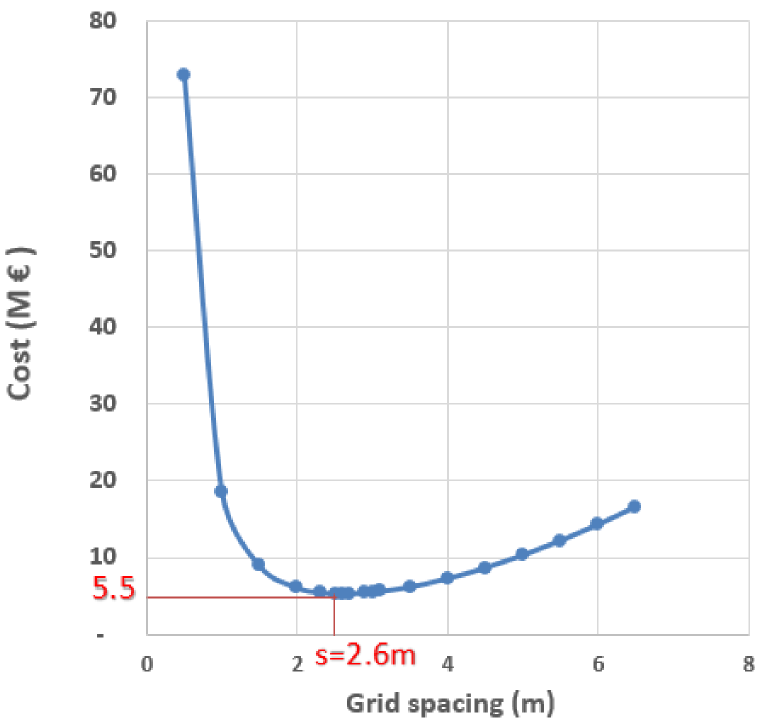
	Time unit rate		Cost unit rate	
Installation	2000	[m/day/unit]	93750	€/unit/day
Equipment	24	[units]		
Material			0.9	[€/m2]
Pumping	17280	[m3/day/unit]	0.04	[€/m3/day]

Note: The index numbers in the above table are based on rough estimations, according to available projects in China. They should be further investigated.

Calculation with optimal Sd= 2.6 m:

Parameter		Unit	Value	
Grid spacing	sd	m	**2.6**	
drainage influence zone	Ad	m2	5.3066	
equivalent drain distance	Dd	m	2.938	
	cv	m2/s	0.0000038	
	ch	m2/s	0.00000418	
consilidation duration	t	s	3675761.321	**42.5** days
	Ls	m	180	
	Bs	m	26.5	
	Ladd	m	25	
required total area	Ad,t	m2	6095	
number of drains	N	-	1149	
required length of drains	Ld	m	57428	
total volume of materials	Vm	m3	230	
pumping volume	Vp	m3	1120188262445	
Total cost		euros	€ **5,285,195**	

Relationship with total costs and grid spacing



INPUT DATA:

Spacing grid	Total cost
0.5	72,892,091
1	18,582,633
1.5	8,951,530
2	6,084,099
2.3	5,469,434
2.5	5,309,248
2.6	5,285,195
2.7	5,292,768
2.9	5,389,919
3	5,474,373
3.1	5,580,005
3.5	6,184,542
4	7,274,786
4.5	8,666,296
5	10,317,564
5.5	12,205,040
6	14,314,556
6.5	16,637,174

Appendix O. Articulation system

Force calculation

ct	0.2		
Lsurge	180	m	
Lsway	26.5	m	
row	1025	kg/m3	
rowair	1.25	kg/m3	
cs	1		retangular shape
uc	1.4	m/s	
uw	20	m/s	assumption
Hs	3.5	m	wave height in normal condition
d	6.4	m	initial draught

wave force

$$F_{wa} = \frac{\rho_w g L}{16}\left((1-c_t)H_{sh}\right)^2$$

sway direction	886.0	Kn
surge direction	130.4	Kn

current force

$$F_c = \frac{\rho_w c_s A_c U_c^2}{2}$$

	A		F	
sway direction	1152	m2	1157.2	kn
surge direction	138.2	m2	138.9	kn

wind force

$$F_{wi} = \frac{\rho_{air} c_s A_w U_w^2}{2}$$

	A		F	
sway direction	2736	m2	684	kn
surge direction	402.8	m2	100.7	kn

Dynamic force

sway direction	1363.6	kn
surge direction	185.0	kn

Total force

sway direction	4090.7	kn
surge direction	555.0	kn

Appendix P. Downstream bed protection for open option

This Appendix presents the detailed calculations of Chapter 10 from the main report.

Appendix P.1 Bed protection directly behind the opening

This part describes the calculation of bed protection directly behind the opening, which corresponds to Section 10.3.4.

Top layer

Protection area division								
A1			A2			A3		
current velocity	7.02	m/s	current velocity	6.3	m/s	current velocity	5.75	m/s
length	150	m	length	200	m	length	50	m
area	58500	m2	area	85000	m2	area	22500	m2
Solution 1								
Material	**Rocks**							
Parameter	unit	value	Parameter	unit	value	Parameter	unit	value
ϕ_{sc}	-	1.5	ϕ_{sc}	-	1	ϕ_{sc}	-	1
Δ	-	1.65	Δ	-	1.65	Δ	-	1.65
Ψ_{cr}	-	0.035	Ψ_{cr}	-	0.035	Ψ_{cr}	-	0.035
k_h	-	0.47	k_h	-	0.56577	k_h	-	0.5423
k_t^2	-	2	k_t^2	-	2	k_t^2	-	1.5
k_{sl}	-	1.5	k_{sl}	-	1.5	k_{sl}	-	1.5
u	m/s	7.02	u	m/s	6.3	u	m/s	5.75
Dn50	m	1.43239	Dn50	m	0.92581	Dn50	m	0.5545
Solution 2								
Material	**Interlocking concrete blocks**							
Parameter	unit	value	Parameter	unit		Parameter	unit	
ϕ_{sc}	-	0.5	ϕ_{sc}	-	0.5	ϕ_{sc}	-	0.5
Δ	-	1.0065	Δ	-	1.0065	Δ	-	1.0065
Ψ_{cr}	-	0.07	Ψ_{cr}	-	0.07	Ψ_{cr}	-	0.07
k_h	-	0.7	k_h	-	0.7	k_h	-	0.7
k_t^2	-	2	k_t^2	-	2	k_t^2	-	2
k_{sl}	-	1.5	k_{sl}	-	1.5	k_{sl}	-	1.5
u	m/s	7.02	u	m/s	6.3	u	m/s	5.75
D	m	0.58288	D	m	0.46945	D	m	0.3911
Solution 3								
Material	**Box gabions**							
Parameter	unit	value	Parameter	unit	value	Parameter	unit	value
ϕ_{sc}	-	0.75	ϕ_{sc}	-	0.75	ϕ_{sc}	-	0.75
Δ	-	0.66	Δ	-	0.66	Δ	-	0.66
Ψ_{cr}	-	0.07	Ψ_{cr}	-	0.07	Ψ_{cr}	-	0.07
k_h	-	0.7	k_h	-	0.7	k_h	-	0.7
k_t^2	-	1.5	k_t^2	-	1.5	k_t^2	-	1.5
k_{sl}	-	1.5	k_{sl}	-	1.5	k_{sl}	-	1.5
u	m/s	7.02	u	m/s	6.3	u	m/s	5.75
D	m	1.00001	D	m	0.8054	D	m	0.6709

Filter design

This part describes the filter design in Section 10.3.5 of the main report.

Area A1	Top layer			layer 1			layer2			layer 3			layer4			Soil		
	dn50	144	cm	dn50	38	cm	dn50	6.4	cm	Designed as 0.25m thick filter mat			Designed as 0.25m thick filter mat					
	thickness	324	cm	thickness	85.5	cm	thickness	30	cm									
	6-10ton			60-300kg			45-200mm											
	d50	171.429	cm	d50	42.000	cm	d50	8.000	cm	d50	1.100	cm	d50	0.200	cm	d50	0.020	cm
	d10min	99.429	cm	d10min	24.360	cm	d10min	4.640	cm	d10min	0.638	cm	d10min	0.116	cm	d10min	0.012	cm
	d10max	144.000	cm	d10max	35.280	cm	d10max	6.720	cm	d10max	0.924	cm	d10max	0.168	cm	d10max	0.017	cm
	d10	121.714	cm	d10	29.820	cm	d10	5.680	cm	d10	0.781	cm	d10	0.142	cm	d10	0.014	cm
	d15min	149.143	cm	d15min	36.540	cm	d15min	6.960	cm	d15min	0.957	cm	d15min	0.174	cm	d15min	0.017	cm
	d15max	162.857	cm	d15max	39.900	cm	d15max	7.600	cm	d15max	1.045	cm	d15max	0.190	cm	d15max	0.019	cm
	d15	156.000	cm	d15	38.220	cm	d15	7.280	cm	d15	1.001	cm	d15	0.182	cm	d15	0.018	cm
	d60max	197.143	cm	d60max	48.300	cm	d60max	9.200	cm	d60max	1.265	cm	d60max	0.230	cm	d60max	0.023	cm
	d60min	180.000	cm	d60min	44.100	cm	d60min	8.400	cm	d60min	1.155	cm	d60min	0.210	cm	d60min	0.021	cm
	d60	188.571	cm	d60	46.200	cm	d60	8.800	cm	d60	1.210	cm	d60	0.220	cm	d60	0.022	cm
	d85min	222.857	cm	d85min	54.600	cm	d85min	10.400	cm	d85min	1.320	cm	d85min	0.360	cm	d85min	0.036	cm
	d85max	264.000	cm	d85max	64.680	cm	d85max	12.320	cm	d85max	1.540	cm	d85max	0.380	cm	d85max	0.038	cm
	d85	243.429	cm	d85	59.640	cm	d85	11.360	cm	d85	1.430	cm	d85	0.370	cm	d85	0.037	cm
				Filter rule			Filter rule			Filter rule			Filter rule			Filter rule		
				stability	2.616	<5	stability	3.364	<5	stability	5.091	<5	stability	2.705	<5	stability	4.919	<5
				int stability	1.549	<10	int stability	1.549	<10	int stability	1.549	<10	int stability	1.549	<10	int stability	1.549	<10
				permeability	4.082	>5	permeability	5.250	>5	permeability	7.273	>5	permeability	5.500	>5	permeability	10.000	>5

Area A2

top layer			layer 1			layer 2			layer3			Soil		
dn50	90	cm	dn50	12.6	cm	dn50	12.8	cm	Designed as 0.25m thick filter mat					
thickness	270	cm	thickness	30	cm	thickness	30	cm						
1-3ton			10-60kg			90-250mm								
d50	103.000	cm	d50	20.000	cm	d50	7.000	cm	d50	1.000	cm	d50	0.020	cm
d10min	59.740	cm	d10min	11.600	cm	d10min	4.060	cm	d10min	0.580	cm	d10min	0.012	cm
d10max	86.520	cm	d10max	16.800	cm	d10max	5.880	cm	d10max	0.840	cm	d10max	0.017	cm
d10	73.130	cm	d10	14.200	cm	d10	4.970	cm	d10	0.710	cm	d10	0.014	cm
d15min	89.610	cm	d15min	17.400	cm	d15min	6.090	cm	d15min	0.870	cm	d15min	0.017	cm
d15max	97.850	cm	d15max	19.000	cm	d15max	6.650	cm	d15max	0.950	cm	d15max	0.019	cm
d15	93.730	cm	d15	18.200	cm	d15	6.370	cm	d15	0.910	cm	d15	0.018	cm
d60max	118.450	cm	d60max	23.000	cm	d60max	8.050	cm	d60max	1.150	cm	d60max	0.023	cm
d60min	108.150	cm	d60min	21.000	cm	d60min	7.350	cm	d60min	1.050	cm	d60min	0.021	cm
d60	113.300	cm	d60	22.000	cm	d60	7.700	cm	d60	1.100	cm	d60	0.022	cm
d85min	133.900	cm	d85min	26.000	cm	d85min	8.400	cm	d85min	1.800	cm	d85min	0.036	cm
d85max	158.620	cm	d85max	30.800	cm	d85max	9.800	cm	d85max	1.900	cm	d85max	0.038	cm
d85	146.260	cm	d85	28.400	cm	d85	9.100	cm	d85	1.850	cm	d85	0.037	cm
			Filter rule			Filter rule			Filter rule			Filter rule		
			stability	3.300	<5	stability	2.000	<5	stability	3.443	<5	stability	4.595	<5
			int. stability	1.549	<10	int. stability	1.549	<10	int. stability	1.549	<10	int. stability	1.549	<10
			permeability	5.150	>5	permeability	2.857	>5	permeability	7.000	>5	permeability	26.321	>5

Area A3

top layer			layer 1			layer2			layer3			Soil		
dn50	59	cm	dn50	21	cm	dn50	12.8	cm	Designed as 0.25m thick filter mat					
thickness	177	cm	thickness	60	cm	thickness	30	cm						
300-1000kg			10-60kg			90-250mm								
d50	70.238	cm	d50	20.000	cm	d50	7.000	cm	d50	0.100	cm	d50	0.020	cm
d10min	40.738	cm	d10min	11.600	cm	d10min	4.060	cm	d10min	0.058	cm	d10min	0.012	cm
d10max	59.000	cm	d10max	16.800	cm	d10max	5.880	cm	d10max	0.084	cm	d10max	0.017	cm
d10	49.869	cm	d10	14.200	cm	d10	4.970	cm	d10	0.071	cm	d10	0.014	cm
d15min	61.107	cm	d15min	17.400	cm	d15min	6.090	cm	d15min	0.087	cm	d15min	0.017	cm
d15max	66.726	cm	d15max	19.000	cm	d15max	6.650	cm	d15max	0.095	cm	d15max	0.019	cm
d15	63.917	cm	d15	18.933	cm	d15	1.988	cm	d15	0.091	cm	d15	0.018	cm
d60max	80.774	cm	d60max	23.000	cm	d60max	8.050	cm	d60max	0.115	cm	d60max	0.023	cm
d60min	73.750	cm	d60min	21.000	cm	d60min	7.350	cm	d60min	0.105	cm	d60min	0.021	cm
d60	77.262	cm	d60	22.000	cm	d60	7.700	cm	d60	0.110	cm	d60	0.022	cm
d85min	91.310	cm	d85min	26.000	cm	d85min	9.100	cm	d85min	0.180	cm	d85min	0.036	cm
d85max	108.167	cm	d85max	30.800	cm	d85max	10.780	cm	d85max	0.190	cm	d85max	0.038	cm
d85	99.738	cm	d85	28.400	cm	d85	9.940	cm	d85	0.185	cm	d85	0.037	cm
			Filter rule			Filter rule			Filter rule			Filter rule		
			stability	2.251	<5	stability	1.905	<5	stability	4.958	<5	stability	2.459	<5
			int. stability	1.549	<10	int. stability	1.549	<10	int. stability	1.549	<10	int. stability	1.549	<10
			permeability	3.376	>5	permeability	9.524	>5	permeability	21.846	>5	permeability	5.000	>5

Appendix P.3 Scour development behind designed bed protection with time

Scour development in time		
Assumption: the velocities vary with time		
protetion length 400m		
	average u'	50.53238557
	omega	0.00014537
t (h)	u (m/s)	hse(m)
0	0	0
0.5	1.487472353	4.710746293
1	2.87367809	6.215867
1.5	4.064244791	7.310350785
2	4.978119077	8.201885679
2.5	5.553084694	8.967630736
3	5.749998177	9.646065692
3.5	5.555453722	10.25956462
4	4.98269585	10.82245304
4.5	4.070717725	11.34453551
5	2.881606509	11.8328597
5.5	1.496316493	12.29268602
6	0.009157754	12.72805999
6.5	-1.478624441	13.14216925
7	-2.865742383	13.53757667
7.5	-4.057761549	13.91637807
8	-4.973529677	14.2803124
8.5	-5.55070158	14.63084076
9	-5.749983592	14.96920435
9.5	-5.557808658	15.29646775
10	-4.987259985	15.61355198
10.5	-4.077180333	15.92125994
11	-2.889527618	16.22029645
11.5	-1.505156837	16.51128408
12	-0.018315485	16.79477584
12.5	1.469772778	17.07126557
13	2.857799406	17.34119629
13.5	4.051268014	17.60496722
14	4.968927661	17.86293951
14.5	5.548304387	18.11544114
15	5.749954422	18.36277095
15.5	5.560149497	18.60520213
16	4.991811469	18.84298517
16.5	4.083632598	19.0763504
17	2.897441397	19.30551013
17.5	1.513993362	19.53066059
18	0.02747317	19.75198356
18.5	-1.460917387	19.96964777
19	-2.84984918	20.18381021
19.5	-4.044764202	20.39461719
20	-4.964313041	20.60220535
20.5	-5.545893121	20.80670251
21	-5.749910666	21.00822844
21.5	-5.562476232	21.20689555
22	-4.996350291	21.40280948
22.5	-4.090074506	21.59606971
23	-2.905347827	21.78676996
23.5	-1.522826048	21.97499871
24	-0.036630785	22.16083958

$$h_s(t) = \frac{(\alpha\bar{u}-\bar{u}_c)^{1.7} h_0^{0.2}}{10\Delta^{0.7}} t^{0.4}$$

$$\alpha\left(\frac{L}{h_0}\right) = 1.5 + (1.57\alpha_{10} - 2.35)e^{(-0.045L/h_0)}$$