MSc Thesis

Reliability Analysis for the Flood Defence System along the Huangpu River, Shanghai

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Reliability Analysis for the Flood Defence System along the Huangpu River, Shanghai

by

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Preface

This thesis signifies the completion of my master period in Hydraulic Engineering, Delft University of Technology. The project has been completed under the supervision and the help of the university and HKV lijn in water.

It is thrilled of me to have had a topic related to the flooding issues in Shanghai, and it was exciting to apply the knowledge I have obtained in the Netherlands to the practice in China. In spite of difficulties and frustration, more often, the joy of overcoming the challenges, of experiencing the atmosphere in HKV, and of improving myself filled the past months.

I would like to thank the members of my graduation committee. Thank Prof. Jonkman, for helping me form the topic, for reading my reports during weekends, and for replying to my emails at late nights. Thank you Fred, for answering my questions and guiding me with great patience. Also, thank Dr. Ke, Dr. Schweckendiek, Mr. Verhagen whose instructions are very precious to me.

Last but not least, I would like to thank HKV lijn in water for offering me such a good opportunity to work with and learn from the amazing colleagues. The 6 months in HKV is a great memory that I will treasure forever.

Zhongqi Wang
Delft, July 18, 2016
Shanghai, a metropolis in the east of China, is vulnerable to flooding (overbank and waterlogging), and numerous floods have happened along the Huangpu River flowing through the city. The floods were mainly caused by the effects of typhoon that contribute to storm surge and strong rainfall, coupled with high tidal level. In order to protect the booming city and nearly 25 million civilians’ lives and properties, a quantity of flood defences have been built along the Huangpu River. Therefore, a trustworthy reliability analysis method is summoned to estimate the flood defence system along the river, which is the main objective of this study.

The thesis presents a reliability analysis method, which was applied to the selected 10-kilometre stretch at the lower reach of the Huangpu River as a case study. As a reference, the VNK2 project of the Netherlands provided the main ideas for developing the method.

The process of the method includes two phases. The first phase is a proof of concept analysis. The reliability analysis was performed for individual segments, and they were subsequently combined into the failure probability of the sub-system (0.31-0.36 per year). In the end of this phase, the weak points of the sub-system (Section 1, Section 2 and Gate a) and the most influential failure mode (overtopping/overflow) were concluded. In other words, the estimated system failure probability was extraordinarily high due to overtopping/overflow with a wave height of approximately 0.7 m.

The second phase is a specific analysis about overtopping/overflow, as it is identified as the most influential failure mode to the sub-system. Wind properties at the study area was therefore specifically analysed. Wind speed, wind direction, and effective fetch length were incorporated into the method. It led to a less conservative failure probability of 0.033-0.055 per year, and the weakest parts among the stretch was expected to start to fail at the water level of around 5.7 m + MSL. Additionally, the influences of typhoon were briefly introduced by observing the track and radius of the typhoon Fung-wong (2014) for example. Typhoon can cause intense variation of wind properties with time, and the assumption that extreme values of different wind properties coincide can lead to the overestimation of the overtopping/overflow probability.

Apart from providing a method for reliability analysis of the flood defence system along the Huangpu River, this study also provides a model to perform probabilistic calculations for the individual components and the system. The tailor-made model is compared with two other common probabilistic models: Prob2B and PC-Ring. This model is more capable in case that the available data are incomplete, and it is highly customisable and scalable for academic researches and applications in other regions of the world.

A more advanced and completed version of the method entails more than two loops. In order to improve the method, recommendations are provided in the final chapter. For instance, floodgates should be studied on prior by taking into account the structural strength of the floodgates and the storage capacity behind; overtopping/overflow probability should be updated by incorporating the effects of rotational typhoon on wind properties; measurements of wind and modeling for waves are recommended along the Huangpu River.
摘 要

上海是中国东部沿海最重要的城市，受到气候变化以及自身城市发展的影响，这座东方都市正遭受着洪涝灾害的威胁。黄浦江贯穿上海市区，这使其首当其冲的成为了城市洪涝灾害的源头之一。台风季节，猛烈的台风裹挟着强降雨。风增水，如若与天文高潮位“三碰头”，则会对整个黄浦江的防洪带来巨大的压力。数十年来，为了确保人民生命财产安全，黄浦江边的防洪系统不断地得到加固、加高，当地政府也为此投入了巨大的人力物力。

因此，黄浦江的防洪系统是否可靠以及其可靠性如何量化成为了本文重点要解决的问题。本文借鉴了已较为成熟的荷兰 VNK2 项目中的洪水风险分析方法，开发出了一套针对黄浦江防洪系统的可靠性分析方法，并选取黄浦江下游段 10 千米长河段，将此方法加以应用。

在本文有限的范围内，此方法共包含两个阶段。第一个阶段是概念分析阶段，这个阶段的目的是全面而概略地完成 VNK2 洪水风险分析方法中的可靠性分析过程。单个防洪墙以及防洪门的可靠性可以通过计算得出，其所构成的 10 千米防洪系统的可靠性也可整合后得出。失效概率为 0.31 - 0.36 每年。此外，通过这个阶段的研究分析，系统中的薄弱段（防洪墙 1、2 以及防洪门 a），造成系统破坏的首要失效机理（漫堤和越浪）也相应地得出。经初步计算，台风季节，黄浦江上的波高可达 0.7 米。

然而，第一个阶段快速得出的结果实际上是不够准确的，于是，第二个阶段是建立在其基础上的具体分析阶段。在本文中，由于漫堤和越浪被证明是首要的失效机理，所以影响该机理的变量在这一阶段中被具体的研究并被引入模型中。这些变量包括了风速、风向、风距、风时。此外，该阶段还以 2014 年登陆上海的台风“凤凰”为例，简单讨论了台风动态特性对计算结果的影响。最后，改进后的模型将系统失效概率更新到 0.033 - 0.055 每年，且系统中最薄弱段在水位达到 5.7 米时已开始出现险情。

除提出分析方法外，本研究也基于 Matlab 提供了一套计算防洪系统可靠性的概率计算模型。在文中，本程序与另外两款常用的计算模型 Prob2B 和 PC-Ring 进行了比较。相比较而言，该程序更加适用于数据资料相对匮乏的研究环境。此外，其基于 Matlab 开发的特性更为其带来了高度的扩展性与灵活性，这一优势在学术研究以及在快速适用于其他案例时能够充分的体现。

当然，一个完善的可靠性分析方法不仅仅只包含这两个阶段。在本文的框架基础上，进一步的研究能够使模型不断的优化，使结果更加精确可靠。本文末尾提出了针对下一阶段研究的建议，例如，通过引入防洪门结构可靠性以及门后蓄水量、防洪门关闭失效的概率应被优先研究：台风动态特性对越浪的影响应被纳入模型。此外，进一步的研究还需要更加可靠、更加完整的风浪数据的支持。

\(^1\) 吴晓辉
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This chapter contains an overview of the study, including its background, its scope and main objectives. In the final section, the structure of this thesis is given.

1.1. Background

1.1.1. Shanghai

Alongside the Yangtze River and the East China Sea, Shanghai is the biggest city in China. It is also one of the busiest and most prosperous metropolises on the Earth. As of 2013, the year-end resident population reached 24.15 million [1].

Geographically, Shanghai is located in the Yangtze River Delta, facing to the Pacific Ocean. As is shown in Figure 1.1, it occupies a central location of China’s east coastline. Thanks to its unique geographic features, it is the most important transport hub in China. Shanghai train station is an essential spot of many main railway arteries. Furthermore, the Shanghai Port is the busiest container ports in the world, which handled 35.28 million TEUs in 2014 (http://www.portshanghai.com.cn).

With a pleasant northern subtropical maritime monsoon climate, Shanghai suffers great rainfall with typhoon in summer. The average annual rainfall is 1,200 millimetres. However, nearly 60% of the precipitation comes during the May-September flood season, which is divided into three rainy periods, namely, the Spring Rains, the Plum Rains and the Autumn Rains.

Superb transport condition provides solid support for the economic development. Shanghai is one of the most important commercial and financial centres in the Asian-Pacific region. A summary of Shanghai’s economic status is quoted from Wikipedia [2]:

*By the end of 2009, there were 787 financial institutions, of which 170 were foreign-invested. In 2009, the Shanghai Stock Exchange ranked third among worldwide stock exchanges in terms of trading volume and sixth in terms of the total capitalisation of listed companies, and the trading volume of six key commodities including rubber, copper and zinc on the Shanghai Futures Exchange all ranked first in the world. In September 2013, the city launched the China (Shanghai) Pilot Free-Trade Zone - the first free-trade zone in main-
1. Introduction

Figure 1.1: The location of Shanghai. It is located in the Yangtze River Delta, facing to the Pacific Ocean. As a coastal city, it occupies a central location of China’s east coast.

land China. The zone introduced a number of pilot reforms designed to create a preferential environment for foreign investment. In April 2014, The Banker reported that Shanghai "has attracted the highest volumes of financial sector foreign direct investment in the Asia-Pacific region in the 12 months to the end of January 2014". In August 2014, Shanghai was named FDi magazine’s Chinese Province of the Future 2014/15 due to "particularly impressive performances in the Business Friendliness and Connectivity categories, as well as placing second in the Economic Potential and Human Capital and Lifestyle categories".

The remarkable economic and industrial status of Shanghai leads to a great importance of a safe environment, because any tiny disaster could result in tremendous consequences.

1.1.2. Huangpu River

The Huangpu River, flowing through Shanghai, is the last significant tributary of Yangtze River before it empties into the East China Sea. The Bund and Lujiazui, the financial and commercial centre of Shanghai, are located along the river. Figure 1.2 shows the location of the river and the condition of water resources.
The Huangpu River is a 113-kilometre-long river with a width of 300-700 meters and an average depth of 9 meters. It runs towards north and ends at the Yangtze River. The average discharge is approximately $180 \text{ m}^3/\text{s}$ [3]. Due to its location, the river is significantly affected by the runoff of the Yangtze River and the tides of the East China Sea. As a result, the Huangpu River experiences remarkable water level changes.

For thousand years, the Huangpu River has been feeding Shanghai with great amounts of fortunes and opportunities. It is the main water source of domestic water supply. On the other hand, the waterway on the river brings great benefits to Shanghai. It supports 60% of import cargo, 98% export cargo of Shanghai, and more than 6 million passengers per year.

### 1.1.3. Definition of Floods

In China, floods are usually classified into two different types: overbank flooding (Hong-Zai) and waterlogging (Lao-Zai) [4]. The former case results from water’s flowing into the defended backland, while waterlogging occurs when the accumulation of precipitation exceeds evapotranspiration and artificial drainage.
The overbank flooding can be identified in different criteria:

- identify by inflow
  The most conservative criterion is to identify a flood once water flowing into the backland. Notice that the quantity of water is not defined precisely in this criteria.

- identify by breaching
  A flood can be identified by whether breaching of the flood defences occur. Breaching can be caused by various failure mechanisms (e.g., overtopping, piping, macro-instability). Once it happened, water would keep flowing into the land until remedial measures were taken.

- identify by consequence
  In the Netherlands, flood risk analysis contains two aspects: reliability analysis and consequence analysis. A flood can be defined by quantifying its consequences, for instance, number of victims and loss of assets, which can be further linked with inundation depth and flow velocity.

Usually, a flood can be defined by quantifying the consequences of the event. However, consequence analysis is not under the scope of this study. Therefore, a flood in this thesis is defined as an overbank flooding event that initiates breaching of the flood defences or excessive amounts of water passing the defence line.

1.1.4. Historical Floods in Shanghai

The Huangpu River with its tributaries and distributaries also brings flood hazard. Moreover, Shanghai is influenced by the monsoon climate. In summer, typhoons with lengthy and intense rainfall invade Shanghai, which are likely to trigger flooding, especially when it is coupled with high astronomical tides. Additionally, Shanghai’s flat and low-lying terrain also increases the flood hazard.

In the long history of the Huangpu River, people had to fight against it. The first record could date back to the year 251 A.D. During the last nearly 1800 years, hundreds of millions of people lost their home and properties. Numerous people even lost their life in the floods. From 1001 A.D. to 1991 A.D., the number of floods was up to 102. According to the record, Flood and drought disasters in Shanghai [5], a severe flooding disaster struck Shanghai in 1962. On 2 August 1962, owing to the typhoon, the highest tidal level reached 4.76m + MSL\(^1\) at the estuary of Suzhou Creek. A large scale of overtopping/overflow occurred with 46 breaches of the flood defences along the Huangpu River. Half of the urban zone was inundated with an inundation depth of nearly 2m, and there had been 10-day drainage before the water was drained out. Transport was interrupted and numerous shops, warehouses, residence were damages. According to statistics, the direct damage was approximately 500 million Chinese Yuans (about 77 million $USD with the present currency).

On 1 September 1981, a typhoon passed Shanghai, which caused regional damage to Shanghai. Under the effects of torrential rainfall, extreme astronomical tide, and typhoon-induced storm surge, the tide level broke the record. It reached 5.74m at Wusongkou\(^2\) and 5.22m at Huangpu Park\(^2\), including a storm surge of 1.24m. In this event, around 10 spots on the flood defences failed owing to breaching, or overflow, or floodgate failure. Part of the urban was inundated. 63 factories halted, and two of them suffered great losses. Throughout the city, 6,790 families experienced inundation of 50cm - 100cm. In

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\(^1\)Hereafter, water levels and geographic elevations are referred to Wusong Datum, and MSL stands for it in this thesis.

\(^2\)Hydrological stations whose locations are introduced in Section 2.3.2.
the urban zone, many warehouses were inundated. 2,400 t grains and 220 t sugar were flooded. The disaster also inflicted on other merchandise such as paper, chemical materials, and cloth, etc [5].

During the last 20 years, the high tidal level along the Hangzhou Bay, the Yangtze River estuary and the Huangpu River has continuously broken history. Since 1981, extremely high tide over 5 meters occurred for 9 times. On 19 August 1997, the high tidal level along the Huangpu River broke the history. At Huangpu Park hydrometric station, the tidal level reached 5.72 m, only 14 cm lower than the tidal level of 1,000-year return period in Shanghai (prescribed in 1984). At the Mishidu station, the tidal level reached 4.27 m, 1 cm higher than the 10,000-year tidal level (prescribed in 1984). Moreover, in the very recent year of 2013, a historical high water level occurred. The water level reached 4.62 m at Mishidu (200-year return period estimated by Dr. Ke in 2014), only 0.10 m lower than the crest level and 1.11 m above the warning water level.

Figure 1.3: Historical water level, compared with the estimated extreme water level. The three lines represent the estimated extreme water levels prescribed in 1984 at the three hydrological stations (Wusongkou, Huangpu Park, and Mishidu)

Figure 1.4: Historical flood events in Shanghai city between 1960-1991. The red stars represent the failure of floodwall, e.g., overtopping, breaching and the failure of floodgates (data source: Yuan et al. 1999; graph designer: Ke, Q., 2014)

\(^3\)since 1949, the recurrence of extreme water level has been estimated officially three times in 1963, 1974, 1982 respectively. Besides, Dr. Ke also estimated the extreme water level in 2014. The changes of the water level condition have been caused by climate change and the rapid development of the area.
Figure 1.3 summarises water levels of some historical flooding events between 1962 and 1992. In addition, the figure 1.4 presents the failure locations of the flood defence system. During the 30 years, the failures occurred mostly along the lower reach. Sometimes, failure was also detected in the tributaries of the Huangpu River, and they were mostly located close to the master river. It may be because the flood defence standard for the tributaries is usually lower than that for the master river, and the water levels at the estuaries are high because of its reaction to the water level at the master river.

Another finding is that the tidal level causing flooding is rising with years. The water level at the Huangpu Park station increased from $4.72 \text{m} + \text{MSL}$ in 1962 to $5.22 \text{m} + \text{MSL}$ in 1981. This implies that the tidal level along the Huangpu River has a heightening tendency, and so does the flooding hazard.

Therefore, the flood risk of Shanghai has been an essential topic, and great damage has been caused by different factors, as is shown in Figure 1.5. According to a recent study [6], Shanghai ranked as one of the top 20 cities in the world in terms of population exposure and economic assets value exposure to the floods. The expected annual risk is estimated as 2,000 people per year in terms of loss of life and 73 million $USD per year in terms of direct economic damage [7].

![Figure 1.5: Direct economic damage by recent floods during 1997-2012 in Shanghai (Ke, Q., 2014)](image)

1.1.5. Flood Defences in Shanghai

In order to reduce the flood risk, structural measures have been taken. According to the report, *Investigation and Evaluation of Dike Defence Capabilities of the Huangpu River in Shanghai* [8], 479.7-kilometre floodwalls (as of 2013), 250 watergates (as of 2006), 1,628 floodgates (as of 2010) and 1,190 tidal gates (as of 2010) were already established.

**Floodwalls**

Floodwalls are permanent hydraulic structures built to protect the back area against flooding. Thanks to their vertical shape, floodwalls are usually applied at locations with scarce space. For example, embankments with historical architecture or commercial zone nearby.

In the Shanghai area, especially alongside the urban reach of the Huangpu River, numerous buildings and facilities are present. Land shortage in the metropolis leads to the wide application of floodwalls and their dominant roles in the flood defences of the Huangpu River. This feature distinguishes the flood defences in Shanghai from that in the Netherlands. In the Netherlands, levees with huge slopes are dominantly applied.
1.1. Background

Geographically, the floodwalls start from Wusongkou to Xihejing on the west side, while on the east bank, it ranges from Wusongkou to Qianbujing. The total length is 479.7 km (2013), 60% of which lies in the urban reach of the floodwalls [8].

Structurally, most floodwalls are a combination of two parts. The lower part is a basement, which functions as a support for the upper part and protection for river banks. The other part, upper part, is usually a vertical wall retaining high water levels and waves.

Along the Huangpu River, the floodwalls are mostly built in reinforced concrete [9]. However, other materials are applied as well. For example, in some parts of the floodwalls (around 100 m) along south Bund, glass are utilised for aesthetic reasons. In general, the glass has a thickness of 18 mm and a height of 1 m above 75 cm concrete wall [10]. Moreover, temporary folder floodwalls made of steel were designed. They could be closed when the warning water level is exceeded.

Figure 1.6: A typical cross section in Bund (Xi & Xu, 2011)

Figure 1.7: Selected cross sections of floodwalls along the Huangpu River in the east bank (Ke, Q., 2014)
Most of the floodwalls in this region are vertical walls loading on revetment basements. Particularly, along the urban reach, many of the floodwalls are innovated for sightseeing and car parking. For instance, in Bund, a typical section ranges from Waibaidu bridge to Xinkai River station with a length of 1,697 km and a width of 17.5 m [9]. A typical cross-section in Bund is shown in Figure 1.6. The sightseeing platform has a crest height of around 4 m, supported by piles. The underneath space is utilised for parking. The pedestrian and carriage path is about 25 m away from the platform. Within this distance, green lands and platforms are connected by ramp way with a level difference of 1-2 meters.

Besides, some other typical cross sections of the floodwalls along the Huangpu River in the east bank are illustrated in Figure 1.7. The design water levels follow the safety standard of 1/1,000 per year (1984). The crest level reduces from 7.2 m + MSL to 6.2 m + MSL from the lower reach to the middle reach [7].

**Floodgates**

Apart from floodwalls, floodgates are another type of essential flood defences. In comparison with floodwalls, floodgates are opened structures, which can be closed once the water level exceeded the warning water level.

The flexibility of floodgates brings benefits. For example, it leaves paths for people and vehicles during the non-flooding period. Ecologically and environmentally, aquatic creatures also benefit from them. Another advantage is that, controllable system provides flexibility to choose between to protect the local backland or to sacrifice it for extra flood storage capacity during extreme floods.

However, floodgates bring more complexity as new failure modes are involved. For example, failure of closure mostly depends on human error. In addition to regular engineering calculations and modelings, more empirical approach (e.g., expert judgement, referring to past documents) should also be considered. Consequently, the results of the study becomes less certain and accurate.

According to official statistics, 1,628 floodgates had been built up till 2010, and 1,557 of them are situated along the urban part of the defence [8]. The total length of these gates are up to 11.3 km [8]. Besides, most of the floodgates are flat gates and mitre gates and are steel structures. Generally, they own a width of 1.5 m - 20 m and a height of 0.6 m - 2.5 m. Typical floodgates are shown in Figure 1.8.
1.1. Background

Potential Safety Hazard

However, in actual operation, potential safety hazard of the flood defences still exists and should be well valued. According to an investigation, main reasons for the dangerous situation include boundary conditions change, illegal operation of vessels, land subsidence and engineering impact caused by projects nearby [11]. Owing to this, it was concluded that the flood defence system along the Huangpu River encounters the following issues:

- Floodwalls
  - insufficient crest height
  - structural unsafety (probability of seepage or structural failure)

- Floodgates
  - insufficient construction standard when gates were designed
  - insufficient heights of gates
  - structural “degradation”

Besides, more information was collected through an interview of Mr. Chen, S. from the Shanghai Water Authority. He added that, nowadays, the flood defences along the Huangpu River are also threatened by failure to close floodgates. This type of failure highly depends on the staff training and closing regime. In the history, it happened occasionally.

1.1.6. Reliability Analysis

“Reliability”, in the calculus of probability, is interpreted as the degree (possibility) in which the subject is able to function properly. This interpretation implies likelihood of a process from good functioning to failure. The paths of failure are termed “failure mechanisms” or “failure modes”; the likelihood of the process is termed “failure probability”.

The state representing the critical threshold from functioning to failure, is a limit state. Therefore, the reliability is the probability that this limit state is not exceeded. To mathematise this threshold, the general form of a reliability function (limit state function) is

\[ Z = R - S \]  

in which:
- \( R \) is resistance or strength against failure
- \( S \) is solicitation or load leading to failure

In different cases, “\( R \)” and “\( S \)” can represent various resistance and solicitation. For instance, in the study for structural failure of dikes. “\( R \)” can stand for the crest level of the dikes, then “\( S \)” should be prescribed as the local water level.

According to the definition of limit state function, the failure probability is

\[ P_f = P(Z \leq 0) = P(S \geq R) \]  

The “reliability” is the probability \( P(Z \geq 0) \) and is therefore the complement of the probability of failure. Mathematically, the expression is

\[ P(Z > 0) = 1 - P_f \]

4Most concepts in this section are quoted from the lecture notes [12]
In order to compute the reliability of an element, the joint committee on structural safety proposed a level-classification of the calculation methods. Three levels are included in the classification:

- **Level III**
  The level III methods (e.g., Monte Carlo method) are full-probability methods. They calculate the probability of failure, by generating numerous stochastic numbers to model the solicitation and resistance variables. The reliability of a target is reflected by occurrence of samplings that are in the state of $Z \leq 0$.

- **Level II**
  Methods of this level (e.g., FORM and SORM calculation) entail linearising the reliability function at a design point. These methods approximate the probability distribution of each variable by a standard normal distribution.

- **Level I**
  By simply taking so-called partial safety factors, the Level I method is considered as a semi-probabilistic method. Strictly speaking, no failure probability is calculated. It’s a design method according to the standards, which is still widely applied in many practical domains.

The term “reliability analysis” stands for reliability-based assessment and design. In this M.Sc. thesis, the “reliability analysis” is a safety assessment procedure for existent flood defences along the Huangpu River. The objective of reliability analysis is to assess the safety level of the structure (or the system).

The structured and organised approach of safety assessment in the Netherlands was initiated in 1996. These old approaches and rules can be characterised as semi-probabilistic method. For the last 20 years, it has been applied and calibrated, based on the temporally available information about the reliability of flood defences.

In the primal assessment approach, the process gives a somewhat binary (pass or failure) outcome to judge whether system is safe or not. Therefore, in recent years, much more insights have been gained about the failure probability of flood defences specific failure mechanisms in the VNK2 project. The focus was switched to failure probability and risk of the system.

### 1.1.7. VNK2 Project

The Flood Risk in the Netherlands 2 project (in Dutch: Veiligheid Nederland in Kaart 2, or VNK2) is aimed at estimating flood risk of major levee systems in the Netherlands, also called dike rings, by calculating both the probability of flooding and its consequences. The starting point and core point of the VNK2 can be summarised into the equation:

$$\text{Risk} = \text{Probability} \cdot \text{Consequence}$$  \hspace{1cm} (1.4)

The flood probability is directly dependent on the strength properties of the flood defence system and the loading on them, while the consequence is a reflection of disasters’ damage on the protected area, for example, Dutch neighborhoods behind dikes and the commercial zone by the Huangpu River.

To evaluate flood risk, as is clarified in the technical background of the VNK2 project [13], it takes the steps as is illustrated in Figure 1.9. To be precise, since the study on the Huangpu River focusses on reliability analysis, steps in “flooding probability analysis” (especially B1, B2) are mainly referred to in the scope of this study.
In this part of VNK2 project, the model PC-Ring was mainly adopted. It has been developed specially to evaluate the reliability of a flood defence system as a composition of segments within a reference period. In this study, PC-Ring was not used as a tool, but the concepts and logics of it were rather useful.

This project in the end provides comprehensive results and advice for the authorities and other relevant stakeholders. For instance, Figure 1.10 reveals the some outcomes of the VNK2 project, in which national risk were mapped. In practice, along with the maps, professional suggestions were also provided regarding the consistence between it and the new safety standard that was proposed by the government. Besides, it also helps prioritise levee reinforcements and develop advanced tools for flood risk analysis [14].

(a) Economic risk  
(b) Individual risk

Figure 1.10: Risk maps for the Netherlands from the VNK2 project (Rijkswaterstaat VNK Project Office, 2014)
Zooming into the reliability analysis, an assessment of the strength of the flood defence system in the Netherlands was done. Instead of judging by either pass or failure, the VNK2 project concluded the failure probability of each failure mode for each dike ring. In Figure 1.11, the levee system 1-13 were taken for instance. It answers the question which parts of the flood defences are more vulnerable and what failure modes are more essential and more worth taking measures for.

![Image](image.png)

**Figure 1.11: Reliability of the flood defences in the Netherlands (levee system 1-13) (Rijkswaterstaat VNK Project Office, 2014)**

### 1.2. Thesis Overview

#### 1.2.1. Problem Definition

The unique climatic and geographic features of Shanghai result in its vulnerability to flooding. In the other hand, its huge and rapidly developing economic scale and high population density could lead to tremendous consequence given flooding. As a result, flood risk of the Huangpu River, and specifically the reliability of its flood defence system, has drawn great attention from researchers, engineers as well as the authorities.

Additionally, international groups (e.g., international insurance companies and research institutes) are also interested in the results of local reliability. Nonetheless, China’s distinct policy of information opacity sets a barrier to them.

With limited access to data and pressing needs for reliability analysis, a proof of concept method is summoned. So the main study problem is defined:

**How can the reliability of the flood defences along the Huangpu River be quantified?**

#### 1.2.2. Research Objectives

The predominant objective of this study is

*to develop a method of estimating the reliability of the flood defences along the Huangpu River.*
In the VNK2 project, researchers proposed a schematised procedure to analyse the reliability of flood defences in the Netherlands, which is adopted as the predominant concept and procedure for this study as well. The steps were shown already in Figure 1.9.

Considering the concepts in VNK2 project and the data deficiency in Shanghai, the tailor-made method for Shanghai includes two phases, namely two loops. The study followed these two loops.

The first loop is a proof of concept phase in which the principle is being quick and general. So, it is supposed to include as many aspects as possible, with however a sacrifice of accuracy and profundity. In this phase, a massive quantity of conservative assumptions have to be made. The aims of this stage is to provide suggestions and proof for further studies in the following loops. Specifically, it has to answer the following questions:

- What failure mechanisms are more vital within the stretch?
- What variables are influential in the limit state functions and are worth further studying on?
- Which parts of the picked stretch are most vulnerable?

In contrast, the second loop (specific analysis) should be less "wide" but more "deep". Through purposeful study, the aim is to improve the accuracy of the results on the basis of the recommendations from the first loop. Therefore, the research question in this phase is the following:

- How can the method be improved to obtain more accurate and more reliable results?

In the end of the first loop, overtopping/overflow was found the most influential failure mode, and wind properties made great contributions to it. Therefore, the goal of the second loop was to specify the variables, i.e., wind speed, wind direction, and fetch, to substitute the conservative and rough assumptions made in the first loop.

In addition, this research is secondarily aiming to quantify the failure probability of flood defences and to improve the VNK2 approach in the respect of its models (e.g., calculation for floodwalls) and its applicability in other areas other than the Netherlands.

1.2.3. Research Scope

As is mentioned, the goal of this study is to develop an approach. Also owing to the limited data availability from Shanghai, the results of reliability can hardly be accurate or qualified for academic or political usage. Instead, it is the approach itself that is worth applying.

The total length of the Huangpu River is up to 113 km, it is of little possibility to assess the entire river within such a short period. Hence, a stretch of about 10 km along the river was selected for a case study. The methods and ideas are also applicable to the rest parts of the system. The stretch selection is introduced in Section 2.2.

Thirdly, the data was preferably adopted from the official database from Shanghai and then the processed data from Dr. Ke, Qian. However, to ensure a fast and overall estimation, the proof of concept method allows assumptions in the case of few available data. Especially in the first loop which is the foundation for further loops, assumption making are permissible and plausible. Furthermore, assumptions are made in every loop. The principle is that the assumptions should be less and less conservative, which implies a lower and lower failure probability of the system. The loops cannot be ceased until the failure probability is decreased to a reasonable and realistic level.
In the precedent sections, flooding is classified into two subtypes. Here, flooding due to overbank by high water level was focussed on. The high water level induced by typhoon-induced storm surge, high astronomical tide and extreme discharge from upper reach of the river was considered. Other effects, for instance, earthquake, urban water-logging were excluded. Besides, note that low water level was not taken into consideration either, for is is beyond the scope of "flooding" although it is also harmful to the flood defences.

Besides, the study is proceeded on the basis of the year 2016. Time as a factor can lead to aging and decay of the flood defence system. Positively, time also brings maintenance and improvement to the system by people. However, they were not tackled in this study although they affect the reliability of the system.

Moreover, applying the principle adopted in the VNK2 project, evacuation measures and emergency remedial measures were not engaged in this study. The actual condition could be much more complex if they were involved.

1.2.4. Outline of Thesis

The thesis proposes an approach of reliability analysis for the flood defences. The sequence of the report chapters is in line with the steps of the approach. Generally, it comprises of two parts, as was explained, proof of concept reliability analysis and specific analysis. In this MSc thesis, the latter phase only contains overtopping/overflow analysis.

The outline of the thesis is demonstrated in flow chart in Figure 1.12. Chapter 1 introduces the background of the study and the formation of the topic. From Chapter 2 to Chapter 6, complete process of reliability analysis is implemented. To be specific, Chapter 2 contains an analysis of the system in which a stretch within the Huangpu River was selected. Moreover, the boundary conditions in the stretch are introduced. Based on it, the stretch was decomposed into segments\(^5\). In chapter 3 and 4, reliability of floodwalls and floodgates are shown respectively. After them, Chapter 5 presents the combination of failure probability of the components\(^5\) in the last chapter of the first loop, results are discussed and suggestions are made for the second loop.

Based on the first loop, the predominant failure mechanism, overtopping/overflow, was focussed on and is shown in Chapter 7. Till then, the academic parts of this thesis are completed. Nevertheless, it does not mean the completion of the reliability analysis for the flood defences along the Huangpu River. In essence, more loops should be proceeded to improve the results. As is mentioned in the research scope, the loops cannot be ceased till a reasonable results. The "reasonable" usually means that the failure probability can be explained physically and is consistent with the local situations.

In the final chapter, conclusions and recommendations are made for further study in the near future. The new loops are expected to follow the 2nd loop.

\(^5\)In this report, "segments" stand for flood defence structures (e.g. floodwall sections or floodgates). They, together with failure mechanisms are termed "components" or "elements".
1.2. Thesis Overview

Figure 1.12: Outline of the thesis

Chapter 1 Introduction

1st Loop:
proof of concept
reliability analysis

Chapter 2 System Analysis

Chapter 3 Floodwall Reliability
- overtopping/overflow
- piping
- structural failure

Chapter 4 Floodgate Reliability
- overtopping/overflow
- piping
- structural failure
- closure failure

Chapter 5 System Reliability

Chapter 6 Discussion and Suggestion

2nd Loop:
specific analysis:
overtopping/overflow

Chapter 7 Overtopping/overflow

Chapter 8 Conclusion and Recommendation
Phase I: Proof of Concept Reliability Analysis
2 System Analysis

2.1. Introduction

As is mentioned in the research scope, a 10-kilometre stretch should be selected for a case study. The selected stretch is introduced, and the reasons of choosing it are presented in the Section 2.2. In the Section 2.3, the boundary conditions within the sub-system are introduced. Finally, the sub-system was decomposed into segments, including floodwall sections and floodgates.

Figure 2.1: Target stretch. Floodgates within the stretch are marked in yellow pins, and the definitions of the Section 1-8 are introduced in the Section 2.4.
2.2. Sub-system Definition

In this thesis, a stretch of 10 km in the lower reach of the river was selected. Hereafter, the selected stretch is also called sub-system. An overview of the sub-system and the reasons of the selection are introduced in this section.

2.2.1. Stretch Introduction

As is shown in Figure 2.1, the selected stretch is located at the eastern bank, lower reach of the Huangpu River. The upper end of the reach is 21 km away from Wusongkou (the river mouth), while the lower end is 11 km away from it. Being close to the river estuary results in significant tidal effects. The total length of the stretch is approximately 10 km, and the river width is about 450 m - 550 m.

The present and future backland use of the stretch determines the significance of this stretch’s role among the entire flood defence system. Moreover, the ownership of floodgates and floodwalls also play a role. According to Figure 2.2, the backlands are mainly developed for industrial use (the northern part), and residential use (the southern part).

Figure 2.2: Land use map of Shanghai urban area. The backlands are mainly developed for industrial use (the northern part), and residential use (the southern part). (Shanghai Municipal Government)
Regarding the conditions of the flood defence system, the sub-system is composed of continuous floodwalls and a number of isolated floodgates. Floodwalls cover a 10 km stretch of the river bank. Moreover, based on approximation, there are totally more than 30 flood gates. However, their exact number, the locations and the structural details are unknown yet.

### 2.2.2. Stretch Choice

Some principles should be taken into account for the choice of stretch, which in other words, provides rational supports for selecting the stretch.

1. The backland use makes the stretch important to the public and the authorities. In this stretch, the backland of the southern parts are mostly developed for commercial and residential use. More importantly, on the south of this backland is Lujiazui, one of the most important CBD (Central Business District) in the world. It gathers large amounts of social fortune and elites. Such an important zone is protected by the famous Bund. Robust concrete vertical walls with about 17.5 m-width platforms are applied. Hence, it is logical to believe that the flood defences in this area are already sufficiently reliable against collapse. Nevertheless, the failure of the selected stretch would cause subsequent damage to Lujiazui if floods spread to there.

2. This stretch raises interests to researchers. There are different types of floodwalls along the stretch leave complexity as well as opportunities. Meanwhile, the northern part is exploited for storage and industries. The variety of operators of floodgates in this part such as regional governments and local industries increases the complexity in analysing the failure of closure.

3. Academic feasibility is another reason. According to the collected data, profiles at only five cross sections are available. Two distinct types of them are included in the selected stretch. With relatively complete and authentic information, more reliable results and recommendations could be obtained.

### 2.3. Boundary Conditions

In this section, boundary conditions within the sub-system is schematised. There are three types of data sources, including the direct data, the presumptions based on the official documents of China, and the assumptions made through reasonable expert judgements. In the first loop, it is inevitable to assume a large portion of data due to limited information and the demands for a quick and simple calculation.

#### 2.3.1. Structural Conditions

The total length of the floodwalls are up to 10 km and over 30 floodgates are present. The actual number of floodgates and profiles were unknown, and only two profiles of floodwalls were collected from Dr. Ke Qian’s doctoral dissertation [7]. Meanwhile, the crest levels of the floodwalls are shown in Figure 2.3.

Additionally, according to the official stipulation for the design of flood defences along the Huangpu River [15], some structural information can be deduced as long as the engineers strictly follow the stipulations. For example, the top level of floodgates should be equivalent to the crest level of adjacent floodwalls. The bottom of floodgates (sill level) should be higher than or equal to the local warning water level.
Figure 2.4: Profiles of representative floodwall cross sections

With the lack of information, multiple assumptions about structural conditions had to be made, based on regional circumstances and expert judgements. The principles are to ensure the assumptions to be consistent with the reality and to be simplified for a proof of concept analysis. These assumptions are listed as follow:

- **Floodwalls**
  - According to the real data, crest levels were measured in hundreds of particular spots, while crest levels of spots in between are presumed by linear interpolation. Crest levels of the sub-system are graphed in Figure 2.3 and are highlighted in red.
  
  - The profiles of the floodwalls along the 10-kilometre stretch were abstracted into the two types of profiles at the two ends of the stretch. Each profile represents for half the selected...
2.3. Boundary Conditions

stretch, also illustrated in Figure 2.3. Figure 2.4a and Figure 2.4b present the two assumed profiles of the floodwalls at the upper reach (Section 1-4) and the lower reach (Section 5-8) respectively.

• Floodgates

– 5 floodgates are assumed to be present, namely, Gate a-e, in Figure 2.1. Their locations are assumed and shown in the map. Precisely, the Gate a-e are respectively 21 km, 19 km, 16 km, 13 km, 11 km away from the river mouth.

– As the width of the gates are very small (usually 1.5 m - 20 m), compared to the length of stretch, they were abstracted into points. However, in a micro scale when analysing the structure of each floodgate, the width is not negligible.

– Except for adjustability, floodgates are assumed to play the same role as floodwalls in the flood defence system. There is no storage capacity behind the floodgates in the proof of concept phase. Hence, if floodgates were opened given the water level exceeding the warning water level, it would be identified as a failure.

– Without sufficient information of floodgates' profiles, the 5 floodgates were assumed to be homogeneous, and their dimensions could be assumed according to Figure 2.5. A comparison between the girl’s height and the gate provides an initial approximation. These dimensions are listed in Table 2.1. The definitions of the items are demonstrated in Figure 2.6.

Figure 2.5: A typical floodgate along the Huangpu River. Approximations were made by comparing the gate with the girl's height.

1 The floodwall section 1-8 are defined in Section 2.4
Table 2.1: Approximations about flood gates. Definition of the items are demonstrated in Figure 2.6.

<table>
<thead>
<tr>
<th>Category</th>
<th>Item</th>
<th>Symbol</th>
<th>Approximation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gate</td>
<td>Top level</td>
<td>$z_c$</td>
<td>= adjacent crest level</td>
</tr>
<tr>
<td></td>
<td>Sill level</td>
<td>$z_s$</td>
<td>= warning water level</td>
</tr>
<tr>
<td></td>
<td>Width</td>
<td>$L_{gate}$</td>
<td>3m - 5m</td>
</tr>
<tr>
<td>Beam</td>
<td>Length</td>
<td>$L_{beam}$</td>
<td>= gate width</td>
</tr>
<tr>
<td></td>
<td>Height</td>
<td>$h_{beam}$</td>
<td>200mm - 400mm</td>
</tr>
<tr>
<td></td>
<td>Width</td>
<td>$b_{beam}$</td>
<td>150mm - 250mm</td>
</tr>
<tr>
<td>Hinge</td>
<td>Number</td>
<td>$n$</td>
<td>At least 4</td>
</tr>
<tr>
<td></td>
<td>Length</td>
<td>$b_{hinge}$</td>
<td>150mm - 200mm</td>
</tr>
<tr>
<td>Other info.</td>
<td>Material and type</td>
<td></td>
<td>Steel flat gate, supported by beams</td>
</tr>
</tbody>
</table>

Figure 2.6: Sketch of a simplified flat floodgate. Red annotations show the definition of dimensions’ notations.

2.3.2. Hydraulic Boundary Conditions

Water levels are the major contributors to the hydraulic loads. Meanwhile, as the study focusses on the situation in typhoon circumstances, the wind-induced wave on the river is also an important factor.

Water Level

Along the Huangpu River, three major hydrological stations are situated respectively at the upper, the middle and the lower reach, namely, Mishidu, Huangpu Park and Wusongkou. Water level data were mainly collected from these three gauge stations. In Figure 2.7, the locations of them are highlighted in blue, while the sub-system is situated between Wusongkou and Huangpu Park. Additionally, it is critical to notice that, the water levels were measured at the west side of the river. It implies that local transversal effects (e.g. wind set-up in the transversal direction, meandering river effects) could result in different water levels at the east side of the river.
Water level information within the stretch was adopted from the conclusion made by Dr. Ke Qian. In her doctoral dissertation, she suggested that, generalized extreme value (GEV) distribution was the preferable extreme value probability distribution at the three stations \(^7\). Water levels at other spots along the river were concluded with an assist of Saint Venant Equations for open channel flow in which river discharge is the main driver of the water level. Besides upstream discharge, the extremely high water level is also a joint contribution of high astronomical tides and typhoon-induced storm surge. In the extreme typhoon conditions, wind set-up could occur, and it contributes to the extreme water level. Hence, these effects were already included in Dr. Ke’s conclusions about the extreme water level condition.

Table 2.2: Extreme water level at the three stations (m + MSL) (Ke, Q., 2014)

<table>
<thead>
<tr>
<th>Gauge station</th>
<th>Return Period (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10,000</td>
</tr>
<tr>
<td>Wusongkou</td>
<td>7.09</td>
</tr>
<tr>
<td>Huangpu Park</td>
<td>6.65</td>
</tr>
<tr>
<td>Mishidu</td>
<td>4.96</td>
</tr>
</tbody>
</table>
In conclusion, according to Dr. Ke, Qian, the extreme water level with return periods at the three stations are listed in Table 2.2, and those along the stretch is summarised in Figure 2.8 and 2.9. Typhoon-induced storm surge and set-up along the river, high astronomical tide and upstream discharge were already incorporated into the water level data. However, transversal effects are not measured in the stations, which are discussed as main factors to cause the uncertainty of local water level.

![Figure 2.8: Extreme water level versus crest level in the sub-system](image1)

![Figure 2.9: Extreme water level versus return periods at floodwall section 1 (19500m - 21000m in Figure 2.8)](image2)

### Absolute Sea Level Rise

Nowadays, the world, especially the coastal cities with low elevation, is undergoing a global sea level rise. According to a research reported in the year 2010, the rate of global sea level rise amounts to...
3.3±0.4 mm/a (between 1993 and 2009) [6]. In Shanghai, a sea level rise has also been witnessed. The rising rate observed in the nearshore of the East China Sea is up to 4.9 mm/a [16].

A relative sea level rise is a combination of absolute sea level rise and land subsidence. It is the perceived sea level rise by human beings. However, the "sea level rise" in this section is the absolute sea level rise. Land subsidence is introduced independently in the next section.

For the completeness of the reliability analysis method, the extreme water level $z_0$ should include an absolute sea level rise:

$$z_0 = \text{extreme water level} + \text{sea level rise (since 2012)}$$  \hspace{1cm} (2.1)

Sea level rise is a time-dependent value, it could exert effects only with a long period of time, for instance, more than 10 years. As is mentioned in the study scope, this research only focuses on the situation at the year of 2016. The water level information was based on the dataset collected till 2012. Therefore, from 2012 to 2016, a 4-year sea level rise amounts to approximately 1.3 cm. The Huangpu River is located inlands, so the actual sea level rise is reduced with distance from the estuary. Hence, when studying on the flood defences along the Huangpu River, effects of sea level rise could be neglected. Therefore, the latter term of Formula 2.1 was assumed to be 0 in this study.

**Wind Wave**

Locally, a typhoon can generate wind waves, coupled with high water level, to pose threats on the flood defences. There is little direct measured wave information available. Similar to the assuming method taken for structures, a deduction can be made according to Technical stipulation to flood defence of the Huangpu River, Shanghai, China: the recommended design wave height is stipulated to be 0.7 m [15].

Compared to water level (5.5 m - 7 m), 0.7 m is an influential value. So, cautious calculations are recommended. In the first stage, a formula from Code for Design of Levee Project (GB 50286-98) was adopted [17]:

$$\frac{gH}{U^2} = 0.13 \tanh \left[ 0.7 \left( \frac{gd}{U^2} \right)^{0.7} \right] \tanh \left\{ \frac{0.0018 \left( \frac{F}{d} \right)^{0.45}}{0.13 \tanh \left( \frac{0.7 \left( \frac{gd}{U^2} \right)^{0.7}}{0.7 \left( \frac{gd}{U^2} \right)^{0.7}} \right)} \right\}$$  \hspace{1cm} (2.2)

in which

- $H$ = significant wave height (m)
- $d$ = average water depth (m)
- $U$ = wind speed at 10 m high (m/s)
- $F$ = fetch (m)
- $g$ = gravitational acceleration (9.81 m/s²)

Alternatively, in the Netherlands, the more popular formulas to compute wave height are Bretschneider formula and Young & Verhagen formula. All the three formulas have similar forms and require similar variables, yet their applicable conditions are diverse. The comparison of the three are discussed later in this section and in Appendix A.1.

In order to utilise the formula, data of wind speed, wind direction, fetch and average water depth were necessary. According to Chorography of Huangpu District, Shanghai, the average water depth is approximately 8 m - 10 m, while the average width is around 500 m [18].

In terms of wind direction, insufficient data was available. Meanwhile, wind directions in typhoon circumstances are rather complex. In the first loop, an assumption was made that, wind was perpendicular
to the structures’ orientations. Note that, with this assumption, impacts of wind are obviously overestimated. Besides, according to the geometrical information given in Figure 2.1, fetch length differs from one another at distinct locations. Generally, fetch ranges from 500 m (river width) to 3,500 m (in the river direction).

Regarding the wind parameters, wind speed and fetch are dependent on the wind direction and the orientation of flood defences. Similar to the solutions for structural boundary conditions, some of real data were collected, while insufficient information was compensated with reasonable deductions and assumptions.

Many results summarised by researchers in China could help make assumptions on the local wind speed. For instance, Mr. Xu, Jialiang concluded in his paper that the average wind speed of typhoons\(^2\) directly attack Shanghai from the East China Sea is about 12.1 m/s, and that the average wind speed of typhoons passing Shanghai from south is around 10.9 m/s [19]. The conclusions were made according to the dataset during the period between 1971 and 2002. He also offered Figure 2.10, which shows the distribution of average wind speed in Shanghai. According to the figures, average wind speed decreases from the downtown to the suburbs. At the sub-system, the average wind speed is 13 m/s - 15 m/s in Figure 2.10a and is 12 m/s - 13 m/s in Figure 2.10b. The average wind speed was conservatively assumed to be 14 m/s. Moreover, the figures also show that from the southern part of the stretch to the northern part, wind speed gets larger. Specifically, wind speed varies from 13.5 m/s in the south of the stretch to 14.5 m/s in the north in Figure 2.10a and increases from 12 m/s to 13 m/s in Figure 2.10b.

\(^2\)Average wind speed is usually defined as the average of 10-minute maximum sustained wind speed.
2.3. Boundary Conditions

Extreme value analysis for 10-minute maximum sustained wind speed\(^3\). Extreme wind speed information with 100-year return periods was collected, and is shown in Figure 2.11. It ranges from 32.5\(m/s\) to 34\(m/s\) (from south to north) [20].

![Figure 2.11: Maximum wind speed distribution map of 100-year return period in lower reaches of the Yangtze River. The sub-system is marked in red. (Huang, S. et al., 2009)](image)

In many cases, average wind speed is selected as a design value if extreme water level (e.g., 1,000-year return period) is decided to be used. However, in this study, typhoon is the main factor causing extremely high water level. Hence, the correlation between extreme water level and extreme wind speed is high. In this loop, the extreme wind speed was adopted, which is 32.5\(m/s\) - 34\(m/s\).

A deterministic calculation was implemented with these mentioned assumptions. By using Formula 2.2 from the China's design code, the wave height reached 0.69\(m\). The assumptions were also adopted in other formulas: Young & Verhagen Formula gave a wave height of 0.73\(m\); Bretschneider Formula did a wave height of 1.21\(m\). The specific formulas and their applicability are discussed in Appendix A.1. Among them, the formula from China's code and the Young & Verhagen Formula returned a consistent value with the design value (i.e., 0.7\(m\)) for flood defence design in Shanghai.

![Figure 2.12: Floodwall section 8 against 10,000-year and 1,000-year water level with 0.7-meter-high wave](image)

What does the 0.7\(m\) wave height mean to the flood defences? Figure 2.12 shows the wind wave striking floodwalls, coupled with 10,000-year and 1,000-year water levels respectively. In the figure, 10-minute maximum sustained wind speed is specified by measuring wind speed at a height of 10 metres for 10 minutes and taking the average and then adopting the maximum value for each typhoon event.
the significance of wind wave can be seen: the high water level plus the high wave flows over the crest although crest level was built higher than the water level.

2.3.3. Geotechnical Boundary Conditions

Soil Property

Shanghai is located in the Yangtze Delta. Under the influence of the Yangtze River, the area is mainly occupied by soft soil. In general, there is a 15m - 20m thick clay layer [21] below which are permeable layers. Unfortunately, detailed information about local soil property was not collected in the first loop. However, an example about soil property was collected and is given in Figure 2.13. The sample was taken near the metro station Yuyuan Road by the Suzhou Creek. Another proof was found from the foundation design code published by Shanghai Urban Construction and Communications Commission [22]. In the appendix of the design code, a recommendation of soil distribution by the Huangpu River is given. It is shown in Table 2.3.

The figure and the table show that silt and clay occupies a large proportion of the soil, and the thickness can exceed 20m. Clay and silt are soft and relatively impermeable, and they are compressible. Moreover, they have less shear strength. These features result in a terrible soil circumstance for constructions in Shanghai.

Backfill soil has a thickness of about 3m - 4m. The property of it is unknown. In essence, the feature of backfill soil to a large extent determines the protection of the structure against overtopping/overflow. Also, its permeability also affects the strength of the floodwall against piping failure.

![Figure 2.13: Example of stratum by the Huangpu River. The sample was taken near the metro station Yuyuan Road by the Suzhou Creek (Chen, F. et al., 2010)](image-url)
2.3. Boundary Conditions

Table 2.3: Recommendation of soil layer distribution by the Huangpu River (Shanghai Urban Construction and Communications Commission, 2010)

<table>
<thead>
<tr>
<th>Layer No.</th>
<th>Soil Description</th>
<th>Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>Brownish or grayish yellow clay</td>
<td>0 - 4</td>
</tr>
<tr>
<td></td>
<td>Gray clayed silt</td>
<td>4 - 15</td>
</tr>
<tr>
<td></td>
<td>Gray sandy silt</td>
<td>4 - 15</td>
</tr>
<tr>
<td></td>
<td>Gray silty sand</td>
<td>4 - 15</td>
</tr>
<tr>
<td>3</td>
<td>Very soft gray silty clay</td>
<td>4 - 15</td>
</tr>
<tr>
<td></td>
<td>Gray sandy silt or silty sand</td>
<td>4 - 15</td>
</tr>
<tr>
<td>4</td>
<td>Very soft gray clay</td>
<td>4 - 20</td>
</tr>
<tr>
<td>5 or 5&lt;sub&gt;1&lt;/sub&gt;</td>
<td>Gray clay</td>
<td>20 - 35</td>
</tr>
<tr>
<td></td>
<td>Gray sandy silt</td>
<td>20 - 35</td>
</tr>
<tr>
<td>5&lt;sub&gt;2&lt;/sub&gt;</td>
<td>Gray silty sand</td>
<td>20 - 35</td>
</tr>
<tr>
<td>5&lt;sub&gt;3&lt;/sub&gt;</td>
<td>Gray or dark gray clay</td>
<td>25 - 40</td>
</tr>
<tr>
<td>6</td>
<td>Dark green or brownish yellow clay</td>
<td>22 - 40</td>
</tr>
<tr>
<td>7&lt;sub&gt;1&lt;/sub&gt;</td>
<td>Straw yellow sandy silty or silty sand</td>
<td>30 - 45</td>
</tr>
<tr>
<td>7&lt;sub&gt;2&lt;/sub&gt;</td>
<td>Gray fine sand with silt</td>
<td>35 - 60</td>
</tr>
<tr>
<td>8&lt;sub&gt;1&lt;/sub&gt;</td>
<td>Gray silty clay with silt</td>
<td>40 - 55</td>
</tr>
<tr>
<td>8&lt;sub&gt;2&lt;/sub&gt;</td>
<td>Gray silty clay interlaid with silty sand</td>
<td>50 - 65</td>
</tr>
<tr>
<td>9</td>
<td>Gray fine, medium or coarse sand</td>
<td>60 - 100</td>
</tr>
</tbody>
</table>

Figure 2.14: Typical floodwall profiles (Dr. Ke, Qian, 2014)

Ground Level and Groundwater Level

Ground level varies at different spots, and there was no data available for the stretch. However, presumption could be made according to Figure 2.14. In the five profiles of floodwalls, ground levels are given. Shanghai is at plain area, the ground level is relatively stable. Therefore, it was reasonable and trustworthy to presume a uniform ground level by averaging the five known values, and it was averagely 5.2m + MSL.
Usually, the groundwater level is equal to the mean river level of the river. When floods occur, the water level rises, and the groundwater level rises correspondingly with a time lag. Impermeable feature of clay layer in Shanghai leads to a slow reaction of groundwater level when flooding. Conservatively, the groundwater level was conservatively assumed to be equal to mean river water level of \( 2.75 \text{ m} + \text{MSL} \) in typhoon seasons (from June to September). The groundwater level was underestimated because the reaction of the groundwater to the rise of outer water level could somehow happen.

### Land Subsidence

In Shanghai, a special boundary condition is land subsidence. Owing to extraordinary usage of groundwater and the construction of infrastructures plus the soft feature of clay, Shanghai is under a threat of land subsidence and its secondary effects. Land subsidence is a time-dependent factor, which could lead to the settlement of flood defences. Consequently, it affects structures in two aspects: crest level could be reduced once structures settled with the ground; unequal land subsidence could cause the deformation of floodwalls. As a result, structural damage would occur. Usually, land subsidence cannot make any difference suddenly. However, for a long time, cumulative land subsidence could pose threats to the flood defences.

Similar to absolute sea level rise, land subsidence was also included in this study. The crest level information was collected in the year of 2008, so until 2016, flood defences have been settling with ground for 7-8 years. Therefore the crest level in the year of 2016 should be equal to collected crest levels minus land subsidence quantities. The general land subsidence was approximated to be \( 8.81 \text{ mm/a} \) [16]. In total, the land subsidence for 7-8 years was about \( 6.2 \text{ cm} - 7.0 \text{ cm} \).

#### 2.3.4. Summary

In summary, Table 2.4 is a list of boundary conditions discussed in this section.

<table>
<thead>
<tr>
<th>Table 2.4: Summary of boundary conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Category</strong></td>
</tr>
<tr>
<td>Structure condition Floodgate</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Hinge</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Other</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Hydraulic condition</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Geotechnical condition</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
</tbody>
</table>
2.4. Sub-system Decomposition

The selected stretch consisted of various types of flood defences, and the strength and load condition of the stretch varied along the stretch. Therefore, the sub-system had to be decomposed into segments. The principle was that, within each segment, the load and strength properties could be assumed to be homogeneous.

Since there were a number of strength and load factors affecting the decomposition, the first step was to choose the predominant factors. This step was accomplished by answering what failure modes were the most influential to the sub-system. In essence, it was an iterative procedure, because the question could not be answered without calculations in the latter steps.

Overtopping/overflow was identified as the predominant failure mechanism in the proof of concept phase, which is shown in the table 5.4. Correspondingly, relevant variables are crest level, water level, wind speed, wind direction, structure type, structure orientation, fetch, and water depth. In the first loop, many of them were presumed to be identical along the stretch, so they wouldn’t make drastic difference even if the stretch was decomposed in diverse ways. As a result, the remaining factors were water level, crest level, structures’ type, structures’ orientation. In the VNK2 project, the similar process was also taken, leading to a typical length of a section of 1km. So, in this study, the length of section started with 1km. For particular parts, longer length was adopted if the properties vary intensely, and vice versa.

![Figure 2.15: Sub-system decomposition. The triangles represent cut-off points for sections. S1, S2, etc. stand for Floodwall Section 1, Section 2, etc.]

As a result, Figure 2.15 illustrates how the decomposition was implemented:

1. The red curve represents the variation of the crest level along the stretch. According to its intensity of variation, 7 cut-off points were defined, complying with the above-mentioned principle.
2. Black lines represent the extreme water level under different return periods. It is noticeable that the water level is quite uniform within the stretch in each condition. Hence, it has limited effects on the ways of decomposing the sub-system, and there is no cut-off point along black lines.

3. Based on the floodwall type, a cut-off point was laid in the middle of the stretch. Meanwhile, each floodgate had to be marked in orange dot so that they were considered as separate segments.

4. According to the orientation of the floodwalls, four cut-off points were defined.

Finally, the sub-system was decomposed into 13 segments, including 5 floodgates and 8 sections of floodwalls. Within each segment, boundary conditions were distinct and entailed schematising. Table 2.5 lists a summary of stretch decomposition and schematised boundary conditions for each segment.

Table 2.5: Summary of sub-system decomposition and boundary conditions of each segment

<table>
<thead>
<tr>
<th>Structure type</th>
<th>Distance from river mouth</th>
<th>Segment</th>
<th>Extreme water level (m + MSL)</th>
<th>Top Level (m + MSL)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floodwall B</td>
<td>11.0km - 13.0km</td>
<td>Section 8</td>
<td>7.02 6.42 6.24 6.00 5.83 5.65 6.63</td>
<td></td>
</tr>
<tr>
<td>(Figure 2.4b)</td>
<td>13.0km - 14.0km</td>
<td>Section 7</td>
<td>7.03 6.42 6.24 6.00 5.82 5.64 6.63</td>
<td></td>
</tr>
<tr>
<td></td>
<td>14.0km - 15.0km</td>
<td>Section 6</td>
<td>7.04 6.43 6.24 6.00 5.82 5.64 6.75</td>
<td></td>
</tr>
<tr>
<td>Floodwall A</td>
<td>15.0km - 16.0km</td>
<td>Section 5</td>
<td>7.04 6.43 6.24 6.00 5.82 5.64 6.72</td>
<td></td>
</tr>
<tr>
<td>(Figure 2.4a)</td>
<td>16.0km - 18.5km</td>
<td>Section 4</td>
<td>7.04 6.43 6.24 6.00 5.81 5.63 6.60</td>
<td></td>
</tr>
<tr>
<td></td>
<td>18.5km - 19.0km</td>
<td>Section 3</td>
<td>7.05 6.43 6.24 5.99 5.81 5.62 6.51</td>
<td></td>
</tr>
<tr>
<td></td>
<td>19.0km - 19.5km</td>
<td>Section 2</td>
<td>7.05 6.42 6.24 5.99 5.81 5.62 6.25</td>
<td></td>
</tr>
<tr>
<td></td>
<td>19.5km - 21.0km</td>
<td>Section 1</td>
<td>7.05 6.42 6.23 5.99 5.80 5.61 6.24</td>
<td></td>
</tr>
<tr>
<td>Floodgate</td>
<td>11.0km</td>
<td>Gate e</td>
<td>7.02 6.42 6.24 6.00 5.83 5.65 6.84</td>
<td></td>
</tr>
<tr>
<td>(Figure 2.6)</td>
<td>13.0km</td>
<td>Gate d</td>
<td>7.03 6.42 6.24 6.00 5.82 5.64 6.84</td>
<td></td>
</tr>
<tr>
<td></td>
<td>16.0km</td>
<td>Gate c</td>
<td>7.04 6.43 6.24 6.00 5.82 5.63 6.87</td>
<td></td>
</tr>
<tr>
<td></td>
<td>19.0km</td>
<td>Gate b</td>
<td>7.05 6.43 6.24 6.00 5.81 5.62 6.51</td>
<td></td>
</tr>
<tr>
<td></td>
<td>21.0km</td>
<td>Gate a</td>
<td>7.05 6.42 6.23 5.98 5.80 5.61 6.26</td>
<td></td>
</tr>
</tbody>
</table>
Reliability of the Floodwall System

In this chapter, the reliability of the floodwall section 1-8 is analysed. The first step is to identify what failure mechanisms play important roles during flooding (Section 3.1). Afterwards, the important failure mechanisms are discussed respectively. In order to build a tailor-made model for the floodwalls, limit state functions of the mechanisms and distributions of corresponding variables were specified.

3.1. Failure Mechanisms

(a) Levee failure mechanisms

(b) Hydraulic structure failure mechanisms

(c) Dune failure mechanisms

Figure 3.1: Failure mechanism in VNK2 project (Vergouwe, 2014)

In the VNK2 project, engineers suggested a number of failure mechanisms which were prevailing among dike rings in the Netherlands. These failure modes include overtopping/overflow, piping, macro-
instability, failure of floodgate closure, dune erosion, etc. Figure 3.1 shows some failure mechanisms for the flood defences in the Netherlands. Among them, overtopping/overflow and piping were predominant, and were focussed on in the VNK2 project.

However, compared with the situation in the Netherlands, that in Shanghai is unique, as hard structures such as floodwalls are mainly used in the flood defence system. Therefore, particular investigations and studies were necessary in this study but VNK2 was a good reference to start with.

Besides, referring to VNK2 project, a strategy to determine failure modes of floodwalls was to capture information from local administrations. Major mechanisms can be extracted from historical events which are usually well collected and organised by local administrations. Moreover, some published reports focussing on the flood defences along the Huangpu River could point out the predominant failure modes directly.

To summarise, the floodwalls along the Huangpu River are mainly under the following threats, which are discussed respectively in this section:

- **Overtopping/overflow**
  Just as is the case in the Netherlands, overtopping/overflow is an important threat to floodwalls in Shanghai. Especially before 1960s, owing to insufficient crest levels, widespread overflow frequently attacked the backland [23]. After 1960s, with heightening of floodwalls, the wide range of overtopping or overflow rarely occurred.

  However, threats of overtopping and overflow still exist, but they usually happened regionally. One reason is the change of hydraulic boundary conditions. In the Huangpu Basin, numerous flood defences were built. As a result, flood capacity of branches in the region were reduced. The water level rose correspondingly. During the period from 1950s to 1990s, the high tidal level at the Huangpu River generally rose by 1.07 m [23].

  Nowadays, according to the information provided by Mr. Chen, S. of Shanghai Water Authority, the heights of floodwalls are able to fulfill the requirement of 1,000-year return period. It ensures that the water level could hardly exceed the crest level. However, overtopping still occurred sometimes.

  Therefore, overtopping is one of the main failure mechanisms for the floodwalls along the Huangpu River and was taken into consideration in this study.

- **Piping**
  In the Netherlands, piping is another critical issue for river dikes. The topographical characteristic, e.g., low ground level, leads to significant water level differences between outer side and inner side of dikes. In addition, earth dikes are the main segments of the Dutch flood defence system. Piping consequently occupies a large proportion of mechanisms to cause dikes’ failure in the Netherlands.

  In comparison, Shanghai also owns low land level. In extreme circumstances, high water levels can be higher than the land level. Meanwhile, as a river affected by tides, the Huangpu River has been undergoing the sea level rise. Besides, Land subsidence and a long-term groundwater over-extraction also increase water level differences.

  However, unlike the situation in the Netherlands, concrete structures with long sheet piles are widely applied beneath the floodwalls along the Huangpu River. An example can be seen in Figure 2.4a. Moreover, an impermeable and cohesive clay/silt layer of up to 20 m also weakens the threats of piping.
3.1. Failure Mechanisms

According to the official reports from Shanghai, piping occurred, but usually didn’t cause severe damages to the floodwalls. In the tide-dominated delta, especially along the lower reach of the Huangpu River where the selected stretch is situated, changeable phreatic line by varying water level mitigates the piping probability [23]. Nowadays, piping would be a problem only when sheet piles rupture or when an extremely high water level lasts for a long period [23]. As a basic type of failure mechanism, piping is discussed. Nevertheless, considering the fact that piping could hardly occur in clay layers. Piping was not expected as a critical problem. In the set-up of the method, it was still simply included, but the result was unreliable and was supposed to be calibrated in further studies.

• Structural Failure

For floodwalls, structural failure could be an essential aspect to study on. In general, there are mainly two types of structural failure, namely, structural instability failure (overall instability failure) and structural strength failure. The former type includes horizontal instability, rotational instability, etc., while the latter mainly behaves as the rupture of structures.

In Shanghai, usually, structural instability issues mostly happened during a low water level. However, in this case, the low water is unable to damage backland areas even if the floodwall failed. Besides, during the low water period, the damaged floodwall could be repaired before the subsequent high tides. On the other hand, during flooding, the back of floodwalls is filled with backfill soil as a support to ensure their stability. Therefore, structural instability was not critical and is not discussed in this report.

By contrast, structural strength failure is a threat. Concrete floodwalls could rupture in high water levels. Besides, more critical triggers are ship collision and adjacent underground or cross-river construction works [11]. These issues could indirectly pose potential threats to structural durability, then structural strength failure becomes more likely to occur during flooding. Besides, these two triggers could impair strength of floodwalls against other failure modes. For instance, underground construction might induce an uneven settlement of floodwalls nearby, which could subsequently deform and rupture. If the issue was not observed, these floodwalls would not be functioning to withstand floods. According to the interview of Mr. Chen, S., this type of failure had occurred seriously once in history.

Mr. Chen also reported that, on average, more than one thousand ship collision events occurred per year. Those events that happen during the period with normal water levels were not taken into account, because the structures usually can be repaired and recover from damage before extreme water levels. The crucial situation is when ships collide the structure, coupled with extreme hydraulic conditions. It was estimated that during a typhoon period (usually 1 day), 3-5 events might occur. Their effects cannot be ignored. In this thesis, ship impacts were not considered, but further studies are recommended.

Therefore, the reliability of floodwalls due to structural strength failure and its failure scenarios are discussed in the section 3.4.

• Macro-instability

Macro-instability failure can happen due to the sliding of outer slopes or inner slopes. The main trigger of the former case is over-loading at the back of the floodwalls and revetment failures. However, usually, these events were more likely to occur under the condition of the low water level, which was beyond the definition of "flooding" in this study.
The instability of inner slopes is another remarkable failure mode, which has been observed in many flooding disasters around the world. However, it was also excluded from the list of failure modes for the Huangpu River. Unlike the Dutch dikes, built in the situation of the lower land level than the outer water level, structures in Shanghai are supported by the massive filling soil behind, pushing them outwards against hydraulic loads.

To summarise, in the study on floodwalls by the Huangpu River, overtopping/overflow, piping, structural strength failure were taken into account. They are discussed in the following sections, but macro-instability, overall instability of structures, and effects of ship collision were excluded from this study.

3.2. Overtopping/overflow

3.2.1. Limit State Function

Overflow/overtopping is a common failure mode for flood defences. Figure 3.2 demonstrates the failure mode impacting on floodwalls. In the graphs, the difference between the two is shown. Overflow is a situation when water level exceeds crest level, while water level plus wave height are loading on the structure when an overtopping event occurs.

\[
Z = q_c - q
\]

When \( z \leq z_c - \Delta l \),
\[
q = m_{os} \cdot \sqrt{g(H)^3} \cdot \exp \left[ -3.0 \frac{(z_c - \Delta l - z_d)}{H} \right] \times 1000
\]  
(3.1)

When \( z > z_c - \Delta l \),
\[
q = \left[ m_{ol} \cdot 0.55 \sqrt{-g(z_c - \Delta l - z_d)^3} + m_{os} \cdot \sqrt{g(H)^3} \right] \times 1000
\]

(3.2)

Figure 3.2: Failure mechanism: overflow/overtopping

In the light of this, a limit state function was defined [24]. It was quoted from PC-Ring model that had been applied in the VNK2 project. In the first loop, these formulas was assumed to be qualified in Shanghai:
in which

\[ q_c = \text{critical discharge} (l/m/s) \]
\[ q = \text{average overflow/overtopping discharge} (l/m/s) \]
\[ H = \text{significant wave height, calculated by Formula 2.2} (m) \]
\[ z_c = \text{crest level} (m) \]
\[ z_0 = \text{local water level} (m) \]
\[ \Delta l = \text{land subsidence} (m) \]
\[ g = \text{gravitational acceleration} (9.81 m/s}^2\]
\[ m_{os} = \text{model factor overtopping } \sim logN(0.34, 0.09^2) \]
\[ m_{ol} = \text{model factor flow } \sim N(1.1, 0.3) \]

In the equations, a critical discharge per unit width and per unit time is introduced. It implies that the threshold of overtopping or overflow failure is not the moment when there is water flow over the structures. Instead, a failure occurs if the discharge of inflow reaches a limit (i.e., \( q_c \)).

The critical discharge is dependent on the backland area protected by the floodwall. The value, for the first loop, is assumed to be a deterministic value of 10\( l/m/s \), referring to a similar case of floodwalls in New Orleans.

### 3.2.2. Distributions and Parameters

In order to run a probabilistic calculation, distributions of the relevant variables had to be analysed. Nevertheless, data limit determined that assumptions had to be made. In the VNK2 project, typical distributions and parameters of some variables were suggested by researchers. So, the strategy was to take VNK2 as a reference and to adjust the parameters by comparing the situation in Shanghai and that in the Netherlands.

#### Model Factor

The limit state functions of overtopping/overflow were quoted from the PC-Ring manual [24], which were utilised in the Netherlands. However, their applicability to the cases in Shanghai had not yet been verified. To leave flexibility to calibrate it, the model factor \( m_{os} \) and \( m_{ol} \) were introduced.

Additional steps have to be taken to determine their distributions. An option could be to determine them by fitting the curve of the functions to the results of field experiments. It was not addressed in this study, and instead, the distributions of them in the PC-Ring were simply adopted, \( m_{os} \sim logN(0.34, 0.09^2) \) and \( m_{ol} \sim N(1.1, 0.3) \).

#### Crest Level

Crest level is a stochastic variable. It varies, dependent on the intensity of variation within each section. The length of section therefore can affect the uncertainty of the crest level. Moreover, construction techniques and quality are also contributors.

Crest Level was assumed to follow the Gaussian distribution, and the mean value for each section was taken according to the Table 2.5. In the VNK2, standard deviation was about 0.1\( m \). In Shanghai, the stretch was also divided into sections of about 1\( km \). Moreover, as the most wealthy and important metropolis in China, Shanghai owns qualified construction quality. Hence, it was reasonable to assume the same standard deviation of 0.1\( m \) as that in the Netherlands.
**Water Level**

The uncertainty of local water level stems from two aspects. On the one hand, the local water level was derived from historical data at the three main hydrological stations. In the process of derivation, errors were inevitable and non-negligible. On the other hand, as is mentioned, the measurements on water levels were processed at the west bank of the river, while the selected stretch is situated at the east side. Transversal hydraulic effects were considered. These effects include transversal set-up by wind and set-up at concave banks.

In the VNK2 project, considering the importance of the water level, great efforts were made to determine the distribution and parameters. However, for the Huangpu River, presumptions were necessary. Wind set-up is often a main contributor to the water level in the Netherlands. However, merely the transversal wind set-up was considered in this study, and it was not significant. It could be proved by using Formula 3.4. Compared to the the water level (5m - 7m) and the wave height of 0.7m, the wind set-up of 0.01m could be ignored.

\[
\Delta z = C_f \frac{U^2}{g d} \cdot \Delta X \approx 0.01 \tag{3.4}
\]

in which:
- \(\Delta Z\) = wind set-up (m)
- \(C_f\) = friction coefficient (≈ 3.75 \times 10^{-6})
- \(U\) = wind speed (≈ 33m/s)
- \(\Delta X\) = half of the river width (≈ 250m)
- \(d\) = average water depth (≈ 9m)

In conclusion, in the first phase, water level was normally distributed. Its mean value was adopted from the Table 2.5. Wind set-up was insignificant, so the uncertainty mainly came from modeling errors and measurement errors. In this study, the standard deviation was assumed to be 0.05m.

**Land Subsidence**

Significant land subsidence is a unique characteristic in Shanghai, which was not emphasised on in the VNK2 project. The magnitude of land subsidence is highly dependent on soil properties and the local groundwater conditions, which vary significantly among different locations. The general land subsidence of the urban area was 0.062m - 0.070m for 8 years. Therefore, a mean subsidence value of 0.065m was assumed and the variation coefficient of it was assumed relatively high to be 0.2. It led to a standard deviation of about 0.01m.

**Wind Speed**

Wind speed and direction during typhoon is rather uncertain. Especially within the urban zone, high-rise buildings increase the roughness against wind. Therefore, in the Netherlands, measurements of wind speed and directions are cautiously executed, and parameters are determined by extreme value analysis. In this study, an approximation was made that wind speed follows normal distribution with a mean value of 33m/s and a standard deviation of 5m/s.

**Fetch**

Fetch length depends on the geometrical interaction between the river and the sections. Wind direction is also a key source of fetch’s variation. Hence, in the Netherlands, fetch length is analysed together with wind directions. As is assumed in Section 2.3, in this loop, wind direction was assumed to be
perpendicular to the floodwalls’ orientations. According to the geometry, the fetch ranged from 500\( \text{m} \) (river width) to 3,500\( \text{m} \). The upper limit could be taken in the extreme case for floodwall section 4 located at the turning point and affected by the wind wave propagating in the river direction (demonstrated in Figure 3.3). Generally, it followed normal distribution with a mean of 2,000\( \text{m} \) and a standard deviation of 500\( \text{m} \).

![Figure 3.3: Demonstration of fetch length for the floodwall section 4](image)

### Average Water Depth

Average water depth is already a numerically processed value, which has little uncertainty. In the PC-Ring model, the standard deviation is set to be 0. However, herein, the reliability of measurement and data source could affect accuracy. So, uncertainty should be considered. As a result, the mean water depth was 9\( \text{m} \) and a small standard deviation was assumed (0.25\( \text{m} \)).

### 3.3. Piping

#### 3.3.1. Limit State Function

Piping is one of the most essential threats on dikes in the Netherlands. For the case of Shanghai, piping is also considered as a potential failure mode. Figure 3.4 shows a typical piping failure. In order to define a limit state function for piping, characteristics of the two typical floodwall structures are discussed firstly.

In Figure 3.5a, the front underground structures (left side in the figure) are sheet piles. Continuous sheet piles are built against seepage. At the right side, square piles are used for structural stability.
penetrating through the soft soil. Basically, square piles have no waterproof function. Both types of piles penetrate through the clay layer, reaching sand layer beneath.

The other type of profile (Figure 3.5b) is a combination of an "L" wall and a slope with revetment. An assumption could be made that the revetments were permeable.

Since the clay layer is up to 15m - 20m, for a rough calculation, piping could only able to occur within the clay layer, even though piping usually could not occur in cohesive layers.

![Figure 3.4: Failure mechanism: piping/seepage](image)

![Figure 3.5: Piping/seepage in Profile A and B](image)

Considering the possible complexity of soil property in Shanghai, a simple function quoted from *Technical Stipulation to Flood Defence of the Huangpu River, Shanghai, China* [15] was adopted as the limit
3.3. Piping

State function:

\[ Z = L - C \cdot \Delta z \]  \hspace{1cm} (3.5)

in which

- \( L = \) seepage length = \( \Sigma L_h + m \Sigma L_v \) (m)
- \( \Delta z = \) water level difference = \( z_0 - z_p \) (m)
- \( L_h = \) horizontal seepage length (m)
- \( L_v = \) vertical seepage length (m)
- \( m = \) conversion coefficient, equal to 2 for sheet piles
- \( z_0 = \) local water level (m)
- \( z_p = \) groundwater level (m)
- \( C = \) coefficient, dependent on soil type and filter layer

3.3.2. Distributions and Parameters

Similar to the steps taken in the analysis of overtopping, distribution and corresponding parameters involved in the limit state function of piping are discussed in this subsection. Water level is already discussed. Besides, groundwater level, seepage length and the coefficient "C" were treated as stochastic variables.

Groundwater Level

As is explained in Section 2.3.3, groundwater level was assumed to equal mean river level during the typhoon season. In fact, along the tide-dominated reach of the Huangpu River, groundwater level at its banks varies significantly. Moreover, the response of groundwater to the river water level is complex, because the fluctuation of the water level is partly induced by the tanglesome typhoon. It also enhances the uncertainty.

In the VNK2 project, the default value of the standard deviation is 0.1 m, which was taken for quantifying the uncertainty of groundwater level. Finally, groundwater level was assumed to follow \( N(2.75 \text{ m}, 0.1 \text{ m}) \).

Seepage Length

Seepage length is a variable associated with the structure profiles of the floodwalls and their seepage path. According to the two representative profiles of floodwalls, a rough assumption could be made that, the mean seepage length was 26 m for the profile A and 12 m for the profile B. Figure 3.5 presents how the seepage length was defined.

Regarding standard deviation, in the Netherlands, a variation coefficient of 0.1 was adopted. In Shanghai, 0.1 could be assumed for the first estimation. Therefore, the standard deviation was about 2.6 m for the profile A and 1 m for the profile B.

Coefficient "C"

In the formula, "C" is a coefficient reflecting the type of local soil and presence of filter layer. In the design stipulation, the "C" ranges from 2 to 3 for clay layer with filter layers and from 3 to 4 for clay layer without filter layers. It is also mentioned that, in the case of sheet piles' presence. Lower bound should be adopted. Therefore, in the case of profile A (without filter layer but with sheet piles), 3 was picked as mean value. In the case of profile B (with filter layer but without sheet piles), 3 was picked as well. Overall, 3 was taken as the mean value of "C" universally for the entire stretch.
In the Netherlands, the Bligh or Lane formula is more common in the PC-Ring model. Similarly, a Bligh constant $C_B$ or a Lane constant $C_L$ were defined. Unfortunately, these two formulas are not applicable for cohesive soil such as clay. The value of $C_B$ and $C_L$ could not be referred to. Nevertheless, for a simple approximation, variation coefficient of 0.15 could be applied for Shanghai, and it led to a standard deviation of 0.45.

### 3.4. Structural Failure

#### 3.4.1. Limit State Function

Structural failure mechanisms of floodwalls include structural instability failure and structural strength failure. As was decided in the discussion about main failure modes of Shanghai, in this case, only structural strength failure was considered. Figure 3.6 presents a typical pattern of the floodwall’s collapse by water pressure under the condition of extremely high water level.

![Figure 3.6: Failure mechanism: structural failure (floodwall). Concrete walls collapse by water pressure under the condition of extremely high water level.](image)

From the sketch, a typical collapse of floodwall is defined as the moment when steels in the concrete rupture due to water-induced bending moment. Considering this, a limit state function simply describes this relation:

\[
Z = M_r - M_s \\
M_r = \frac{0.87f_y h_{wa} A_s}{1.7} \times \frac{1}{1000} \\
M_s = \frac{1}{6} \rho_w g (z_0 - (z_1 - \Delta l))^3
\]  

in which

- $M_r =$ resistant moment $(N \cdot m)$
- $M_s =$ moment loading on the wall $(N \cdot m)$
- $A_s =$ actual steel sectional area used in the floodwall $(mm^2)$
3.4. Structural Failure

\[ \rho_w = \text{water density (1,000} \, \text{kg/m}^3) \]
\[ g = \text{gravitational acceleration (9.81} \, \text{m/s}^2) \]
\[ z_0 = \text{water level (m + MSL)} \]
\[ z_t = \text{ground level (m + MSL)} \]
\[ \Delta l = \text{land subsidence (m)} \]
\[ f_y = \text{tensile strength of steels (N/mm}^2) \]
\[ h_{\text{wall}} = \text{thickness of the floodwall (mm)} \]

In the equation, the limit state was defined as the moment when critical bending moment is equal to the actual loading caused by water pressure. The critical moment is influenced by the quantity and type of steel bars in the concrete and the geometry of the walls.

Another striking sub-mechanism of structural failure is caused by ship collision. The limit state function can be built by referring to PC-Ring:

\[
Z = E_c - E_0
\]
\[
E_0 = m_E \cdot \frac{1}{2} mv^2
\]

in which
\[ E_c = \text{allowable energy (kN} \cdot \text{m/s}^2) \]
\[ E_0 = \text{energy caused by collision (kN} \cdot \text{m/s}^2) \]
\[ m_E = \text{model factor} \]
\[ m = \text{mass of the ship (ton)} \]
\[ v = \text{collision speed (m/s)} \]

According to the limit state function, the probability of it depends on the mass and speed of the vessels and the strength of the structures. Apart from them, the occurrence frequency of collision is also an essential variable. However, the mechanism was not tackled specifically in this thesis. As a recommendation, it can be furthered in the future, but the limit state function has to be adjusted to the actual situation of vessels and structures along the Huangpu River.

3.4.2. Distributions and Parameters

In the limit state function, water density and gravitational acceleration are assumed to be deterministic value. Moreover, the distributions and the corresponding parameters of water level and magnitudes of land subsidence were already defined. Apart from them, four unique variables have to be defined in this section, namely, ground levels, actual steel sectional area in floodwalls, tensile strength and thickness of floodwalls. Again, assumptions had to be made for them.

Wall Thickness

According to the information acquired from the five typical profiles (Figure 2.14, the thickness of floodwalls at the top is about 300mm - 400mm. According to the design code for flood defences in Shanghai [15], 400mm is the minimum thickness of the main parts of the floodwalls. Therefore, a conservative presumption was be made that mean thickness is about 400mm.

Uncertainty of thickness is mainly caused by structural type and construction quality. Structural type was assumed to be homogeneous along the Section 1-4 and 5-8 respectively. Meanwhile, construction quality could be guaranteed in Shanghai for such an important project. As a result, the standard deviation was relatively small, which was assumed to be 25mm.
3. Reliability of the Floodwall System

Ground Level

The ground level was discussed in the section about geotechnical boundary conditions. The mean value is around 5.2 m + MSL. The standard deviation of it depends on backland utilization and land subsidence. Since in this study, land subsidence was taken as an independent stochastic variable, variation of ground level only depended on the backland use. In a conservative assumption, 0.1 m was adopted as its standard deviation.

Steel Tensile Strength and Sectional Area

Owing to the limited information about steel arrangement in the floodwalls, rough assumptions had to be made. Note that, thanks to the high production quality of steel and construction capability, standard deviations were supposed to be quite small for these two variables. In consequence, steel tensile strength followed normal distribution with a mean value of 360 N/mm² and a standard deviation of 10 N/mm². Steels were assumed to be placed in Φ10-200 mm, so the sectional area of it followed N(393 mm², 10 mm²).

3.5. Summary

In summary, the floodwalls could fail mainly due to overtopping/overflow, piping, structural collapse. Limit state functions were defined and distributions of involved variables were assigned. The table 3.1 is a list of all the relevant variables and their distributions.

Table 3.1: List of variables: floodwall

<table>
<thead>
<tr>
<th>Variable</th>
<th>Unit</th>
<th>Distribution</th>
<th>Mean</th>
<th>Standard deviation</th>
<th>Variation coefficient</th>
</tr>
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<tbody>
<tr>
<td>mₘₜ</td>
<td>-</td>
<td>Lognormal</td>
<td>0.34</td>
<td>0.009</td>
<td>-</td>
</tr>
<tr>
<td>mₘ₀</td>
<td>-</td>
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<td>1.1</td>
<td>0.3</td>
<td>0.27</td>
</tr>
<tr>
<td>zₑ</td>
<td>m+MSL</td>
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<td>Table 2.5</td>
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<td>0.015</td>
</tr>
<tr>
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<td>m+MSL</td>
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<td>Table 2.5</td>
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<td>0.01</td>
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<td>0.1</td>
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</tr>
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</tr>
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</tr>
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<td>0.25</td>
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<td>0.03</td>
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<td>-</td>
<td>-</td>
</tr>
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<td>1.2</td>
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<tr>
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<td>-</td>
<td>-</td>
</tr>
<tr>
<td>g</td>
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<td>-</td>
<td>-</td>
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<tr>
<td>fₑ</td>
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</table>
4

Reliability of Floodgates

A floodgate is also a water retaining structure. It is generally open for the passage of civilians and vehicles, and it has to be shut when the water level exceeds the warning water level. Usually, the warning water level is prescribed to be equal to the sill level of the floodgate.

In this chapter, the focus is shifted from the floodwalls to the floodgates. Five hypothetic floodgates, namely Gate a-e, were defined within the stretch. Like the steps taken for floodwall analysis, the first step was to define major failure mechanisms that could trigger damages to the floodgates. Afterwards, these failure modes are discussed respectively in the following sections.

4.1. Failure Mechanisms

![Figure 4.1: Fault tree: floodgate.](image)

The failure modes taken into account in this study are marked in red.

Considering similarities between floodgates and floodwalls, overtopping/overflow and piping were considered in the evaluation for the floodgates. The limit state functions, relevant variables and their pa-
4. Reliability of Floodgates

Parameters could be applied as well for the floodgates. Note that in the case of piping at the floodgates, information was of no sufficiency. An assumption had to be made that the underground structure and consequently the failure probability of piping was equal to that of the adjacent floodwalls.

However, the adjustable feature of floodgates also brought extra hazards to the study. Failure due to non-closure was taken into account. Besides, due to their structural differences, structural failure had to be redefined for floodgates.

As a summary, Figure 4.1 presents the fault tree for the floodgates. In the tree, the failure modes taken into account in this study are marked in red. Failure due to non-closure, as was mentioned, is a unique failure mode of floodgate. It could be triggered as long as the gates physically failed to close (i.e., "failure of closure") AND the amount of inflow exceeded the strength behind the gates (i.e., "failure by inflow"). Specifically, "failure of closure" would be triggered when both the closing process failed AND the remedial measures failed. Regarding the latter condition, the "strength" was defined to be the structural strength against collapse OR allowable storage capacity against inundation.

- **Structural Failure**
  If a water level exceeded the local warning water level, the floodgate would have to be shut. Then water level differences between inside and outside of the gate would load on the steel floodgate. If the loads were sufficiently high, beams (horizontal support) or rear posts (vertical support) could not function, and the failure would eventually occur.

  According to the investigation on the floodgates of the Huangpu River [8], there existed a threat of structural failure during floods. A main cause is corrosion of steels owing to aging. In the year of 2012, a field investigation was implemented and 56 floodgates were found structurally in danger (about 3% of the total) [8]. Therefore, structural failure was ought to be taken into consideration.

- **Closure Failure**
  Floodgates can be closed at high water levels. However, the failure of closure signifies a huge gap within a flood defence system through which water flows in, and its consequence is severe.

  Furthermore, the triggers of closure failure is rather complex. It depends not only on technical aspects but also on the closure regime and staff training. As a result, it was worthwhile to incorporate closure failure in the method.

### 4.2. Structural Failure

#### 4.2.1. Limit State Function

Structural failure of floodgates, in this case, was defined as the case when the supports ruptured due to water head differences, and floodgate, as a result, could not sustain their function. The supports could be the beams horizontally supporting the floodgates against opening or vertical rear posts (e.g., hinges). The limit state functions for both of them are discussed respectively.

**Horizontal Beams**

In the sketch of a simplified flat gate, Figure 2.6, three beams support the floodgate. With hydraulic loads, it is reasonable to believe that the lowest beam has to withstand the majority of loads. Figure 4.2 shows the rupture of a beam. A limit state function was built on the state of the lowest beam’s rupture
4.2. Structural Failure

under water pressure:

\[ Z = f_r - f_s \]  
\[ f_s = \frac{3\rho_w g(z_0 - z_s)l_{beam}^2}{4h_{beam}^2 \times 10^6} \]  

in which

- \( f_r = \) tensile strength of steel used on the floodgate (N/m²)
- \( f_s = \) tensile stress on the floodgate (N/m²)
- \( \rho_w = \) water density (1,000 kg/m³)
- \( g = \) gravitational acceleration (9.81 m/s²)
- \( z_0 = \) water level (m + MSL)
- \( z_s = \) sill level (m + MSL)
- \( l_{beam} = \) beam length (mm)
- \( h_{beam} = \) beam height (mm)

**Figure 4.2: Failure mechanism: structural failure (floodgate)**

### Hinges and Bolts

The water pressure loading on the floodgate eventually converges into pull force loading on the hinges. The hinges are connected with rear walls by bolts which the pull force ultimately loads on. By assuming a resistant force of each bolt, a limit state function could be defined:

\[ Z = F_r - F_s \]  
\[ F_r = A_b \cdot f_b = \frac{1}{4} \pi D^2 \cdot f_b \]  
\[ F_s = \frac{1}{2} \rho_w g(z_0 - z_s)^2 L_{gate} \]  

in which

- \( F_r = \) strength of a single bolt (N)
\[ F_b = \text{pull force loading on a single bolt (N)} \]
\[ A_b = \text{cross-sectional area of a single bolt (mm}^2) \]
\[ D = \text{cross-sectional diameter of a single bolt (mm)} \]
\[ f_b = \text{tensile strength of a single bolt (N/mm}^2) \]
\[ \rho_w = \text{water density } (1,000 \text{kg/m}^3) \]
\[ g = \text{gravitational acceleration } (9.81 \text{m/s}^2) \]
\[ z_0 = \text{water level (m + MSL)} \]
\[ z_s = \text{sill level (m + MSL)} \]
\[ L_{gate} = \text{width of the floodgate, equal to } L_{beam} (m) \]
\[ n_h = \text{number of hinges} \]
\[ n_b = \text{number of bolts per hinge} \]

### 4.2.2. Distributions and Parameters

Note that the information about the floodgates was nearly blank except for their material type and water levels in front of the gates. So, the results of probabilistic calculation would not be accurate.

In Appendix A.2, a deterministic calculation was performed, and it was concluded that the structural failure could hardly occur.

However, for the sake of a complete reliability analysis method, structural failure were still taken into account, and the results would be more trustworthy once better data were collected or better limit state functions were defined. Consequently, a number of assumptions for the variables had to be roughly made by using common knowledge.

#### Sill Level

According to the design stipulation, the sill levels of the floodgates could be presumed to be equal to the local warning water level [15]. In the stipulation, warning water levels of the three main hydrological stations were given. Local warning water levels could be derived by linear interpolation.

Uncertainty of sill levels stem from construction error and structural aging. However, to guarantee the floodgates’ function, sill should be accurately constructed, otherwise the gates could not close or retain water. Therefore, it was reasonable to assume a small standard deviation. The Table 4.1 lists mean values and standard deviations of the sill levels. Compared with the local extreme water level, sill levels are approximately equal to the water level with 1-year return period. Therefore, once the water level exceeded the 1-year water level, the floodgates have to be closed.

**Table 4.1: Assumed sill levels of the floodgates**

<table>
<thead>
<tr>
<th>Location</th>
<th>Distance to Wusongkou (m)</th>
<th>Mean value (m + MSL)</th>
<th>Standard deviation (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wusongkou</td>
<td>0</td>
<td>4.80</td>
<td>-</td>
</tr>
<tr>
<td>Gate e</td>
<td>11,000</td>
<td>4.69</td>
<td>0.05</td>
</tr>
<tr>
<td>Gate d</td>
<td>13,000</td>
<td>4.68</td>
<td>0.05</td>
</tr>
<tr>
<td>Gate c</td>
<td>16,000</td>
<td>4.65</td>
<td>0.05</td>
</tr>
<tr>
<td>Gate b</td>
<td>19,000</td>
<td>4.62</td>
<td>0.05</td>
</tr>
<tr>
<td>Gate a</td>
<td>21,000</td>
<td>4.60</td>
<td>0.05</td>
</tr>
<tr>
<td>Huangpu Park</td>
<td>26,000</td>
<td>4.55</td>
<td>-</td>
</tr>
</tbody>
</table>
4.3. Failure of Closure

Steel and Bolt Tensile Strength

The distribution and relevant parameters of steel strength depend on the choice that designers made and the quality of products. Since no such information was available, an assumption was made that it followed normal distribution with a mean of \(300 \text{N/mm}^2\) and a standard deviation of \(10 \text{N/mm}^2\). Besides, the same assumption was also taken for the tensile strength of a single bolt.

Beam Height and Length

The definition of beam height and length is graphed in Figure 2.6. Assumptions were made that \(h_{\text{beam}} \sim N(300\text{mm}, 20\text{mm})\) and \(L_{\text{beam}} \sim N(4000\text{mm}, 100\text{mm})\).

Gate Width

Gate width was assumed to be equal to the beam length, so it followed a normal distribution with a mean of \(4,000\text{mm}\) and a standard deviation of \(100\text{mm}\).

Cross-sectional Diameter of Bolts

Similarly, cross-sectional diameter depends on the type of bolts that adopted by engineers, which was unknown. However, a simple assumption could be made that the cross-sectional diameter had a mean of \(30\text{mm}\). The standard deviation depends on the quality of bolts, which can be guaranteed. Hence, it was assumed to be \(0.3\text{mm}\).

Number of Hinges and Bolts

These two variables also depend on the design. An observation from Figure 2.5 suggested that there could be 8 hinges for a gate. Nevertheless, from the figure, the number of bolts could not be observed. A conservative assumption was made that in each hinge, there might be 4 bolts. Besides, the number of bolts and hinges were fixed once the gates were built, so it is a deterministic value.

4.3. Failure of Closure

Closure failure is a unique issue of floodgates. According to the assumption made in the preceding chapter, there assumed to be no storage capacity at the back of the floodgates. It implied that failure was defined as the moment when a large amount of water flowed through the gates given non-closure. Considering the fault tree shown in the beginning of this chapter, the closure failure simplified as a single event, "failure due to non-closure", and a failure probability of 1/100 per event was assumed. Note that, closure failure could only occur under the condition when the water level exceeded the sill level.

Closure failure could occur for complex reasons, and a technical limit state function was difficult to define. Floodgates along the Huangpu River can be shut automatically or manually. Besides breakdown of shutting system and human error are also triggers, which add more uncertainty to the gates’ behavior. Occurrence of human-induced closure failure is dependent on the training degree of operators. Besides, the closing regime of the floodgates may affect shutting. For instance, desertions during the floods might cause severe consequence.

In the first loop, a conditional failure probability of 1/100 per gate could be assumed for closure failure. Since human errors would not vary much under different water level conditions, it was also plausible
to assume the 1/100 for all the conditions as long as the sill level was exceeded. So, the probability of closure failure could be formulised:

\[ P(\text{Closure failure} \mid z_0 > z_s) = 1/100 \text{ per event per gate} \]  \hspace{1cm} (4.6)

in which

\[ z_0 = \text{water level (m + MSL)} \]
\[ z_s = \text{sill level (m + MSL)} \]

### 4.4. Summary

In summary, floodgates could fail owing to overtopping/overflow, piping, structural failure that was defined as the rupture of lowest beam or vertical rear posts, and closure failure (conditionally 1/100 per event per gate). Assumptions were made for the probabilistic calculation, which are listed in Table 4.2.

**Table 4.2: List of variables: floodgate**

<table>
<thead>
<tr>
<th>Variable</th>
<th>Unit</th>
<th>Distribution</th>
<th>Mean</th>
<th>Standard deviation</th>
<th>Variation coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>( m_{o.s} )</td>
<td>-</td>
<td>Lognormal</td>
<td>0.34</td>
<td>0.009</td>
<td>-</td>
</tr>
<tr>
<td>( m_{ul} )</td>
<td>-</td>
<td>Normal</td>
<td>1.1</td>
<td>0.3</td>
<td>0.27</td>
</tr>
<tr>
<td>( z_c )</td>
<td>( m + \text{MSL} )</td>
<td>Normal</td>
<td>Table 2.5</td>
<td>0.1</td>
<td>0.015</td>
</tr>
<tr>
<td>( z_0 )</td>
<td>( m + \text{MSL} )</td>
<td>Normal</td>
<td>Table 2.5</td>
<td>0.05</td>
<td>0.01</td>
</tr>
<tr>
<td>( z_l )</td>
<td>( m + \text{MSL} )</td>
<td>Normal</td>
<td>5.2</td>
<td>0.1</td>
<td>0.02</td>
</tr>
<tr>
<td>( z_p )</td>
<td>( m + \text{MSL} )</td>
<td>Normal</td>
<td>2.75</td>
<td>0.1</td>
<td>0.04</td>
</tr>
<tr>
<td>( z_s )</td>
<td>( m + \text{MSL} )</td>
<td>Normal</td>
<td>Table 4.1</td>
<td>0.05</td>
<td>0.01</td>
</tr>
<tr>
<td>( \Delta l )</td>
<td>( m )</td>
<td>Normal</td>
<td>0.065</td>
<td>0.01</td>
<td>0.15</td>
</tr>
<tr>
<td>( U )</td>
<td>( m/s )</td>
<td>Normal</td>
<td>33</td>
<td>5</td>
<td>0.15</td>
</tr>
<tr>
<td>( F )</td>
<td>( m )</td>
<td>Normal</td>
<td>2,000</td>
<td>500</td>
<td>0.25</td>
</tr>
<tr>
<td>( d )</td>
<td>( m )</td>
<td>Normal</td>
<td>9</td>
<td>0.25</td>
<td>0.03</td>
</tr>
<tr>
<td>( q_c )</td>
<td>( l/m/s )</td>
<td>Deterministic</td>
<td>5</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>( C )</td>
<td>-</td>
<td>Normal</td>
<td>3</td>
<td>0.45</td>
<td>0.15</td>
</tr>
<tr>
<td>( L(a, b, c) )</td>
<td>( m )</td>
<td>Normal</td>
<td>26</td>
<td>2.6</td>
<td>0.1</td>
</tr>
<tr>
<td>( L(d, e) )</td>
<td>( m )</td>
<td>Normal</td>
<td>12</td>
<td>1.2</td>
<td>0.1</td>
</tr>
<tr>
<td>( p_w )</td>
<td>( k\gamma/m^3 )</td>
<td>Deterministic</td>
<td>1,000</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>( g )</td>
<td>( m/s^2 )</td>
<td>Deterministic</td>
<td>9.81</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>( f_b, f_h )</td>
<td>( N/mm^2 )</td>
<td>Normal</td>
<td>300</td>
<td>10</td>
<td>0.03</td>
</tr>
<tr>
<td>( h_{beam} )</td>
<td>( mm )</td>
<td>Normal</td>
<td>300</td>
<td>20</td>
<td>0.06</td>
</tr>
<tr>
<td>( L_{beam, l_{gate}} )</td>
<td>( mm )</td>
<td>Normal</td>
<td>4,000</td>
<td>100</td>
<td>0.025</td>
</tr>
<tr>
<td>( D )</td>
<td>( mm )</td>
<td>Normal</td>
<td>30</td>
<td>0.3</td>
<td>0.01</td>
</tr>
<tr>
<td>( n_b )</td>
<td>-</td>
<td>Deterministic</td>
<td>4</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>( n_h )</td>
<td>-</td>
<td>Deterministic</td>
<td>8</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
In this section, the reliability of each section and each failure mechanism is combined into the reliability of the entire sub-system. To begin with, fault trees of flood defences applied in the PC-Ring are introduced in Section 5.1. In the following sections, results for each segment and failure mode by probabilistic calculation are summarised and combined into failure probability for the entire sub-system. Conditional and unconditional failure probability are introduced respectively.

5.1. Fault Tree

In the VNK2 project, failure modes of the hydraulic structures were schematised in fault trees. After being combined with the actual situation in Shanghai, they are modified and presented in Figure 5.1 - 5.3. Among all the components, those that were considered and included in the first loop calculation are marked in red.

As is shown in Figure 5.1, the stretch is a serial system in which the system would fail when any of the segments (Section 1-8 or Gate a-e) fail.

In the Figure 5.2 and 5.3, floodwall section 1 and floodgate a were respectively taken for example. Different from the original version of fault trees applied in the VNK2 project, structural failure mechanism in Shanghai does not include overall instability.

Figure 5.1: Fault tree: segments
5.2. Tailor-made Model

A customized model is built on the probabilistic toolbox in OpenEarth. It provides First Order Reliability Method (FORM) and Monte Carlo Simulation (MCS). The two methods are introduced in Appendix B. MCS was selected to use in this study, because it can provide relatively accurate results once the number of samples was sufficiently large. FORM doesn’t work well especially when operating on the overtopping/overflow formula 3.1. At the threshold of whether the water level exceeds the crest level, the failure mechanism was specified into two formulas for overtopping and overflow respectively (Formula 3.2 and 3.3). Therefore, the design point continuously “jumps” between two functions, and in practice, the model ultimately identified it as a "non-convergence".

The model was developed in Matlab in which the number of samples can be specified for MCS. Ad-
5.2. Tailor-made Model

Additional scripts were customised in order to perform bulky calculations for all the failure modes and segments in various water level conditions.

<table>
<thead>
<tr>
<th>Section</th>
<th>1000yr</th>
<th>1000yr</th>
<th>500yr</th>
<th>200yr</th>
<th>100yr</th>
<th>50yr</th>
<th>Crest</th>
<th>Sill</th>
<th>L</th>
<th>sd_sill</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section_1</td>
<td>7.05</td>
<td>6.42</td>
<td>6.23</td>
<td>5.99</td>
<td>5.80</td>
<td>5.61</td>
<td>6.24</td>
<td>0.00</td>
<td>26.00</td>
<td>0</td>
</tr>
<tr>
<td>Section_2</td>
<td>7.05</td>
<td>6.42</td>
<td>6.24</td>
<td>5.99</td>
<td>5.81</td>
<td>5.62</td>
<td>6.25</td>
<td>0.00</td>
<td>26.00</td>
<td>0</td>
</tr>
<tr>
<td>Section_3</td>
<td>7.05</td>
<td>6.42</td>
<td>6.24</td>
<td>5.99</td>
<td>5.81</td>
<td>5.62</td>
<td>6.51</td>
<td>0.00</td>
<td>26.00</td>
<td>0</td>
</tr>
<tr>
<td>Section_4</td>
<td>7.04</td>
<td>6.43</td>
<td>6.24</td>
<td>6.00</td>
<td>5.81</td>
<td>5.63</td>
<td>6.80</td>
<td>0.00</td>
<td>26.00</td>
<td>0</td>
</tr>
<tr>
<td>Section_5</td>
<td>7.04</td>
<td>6.43</td>
<td>6.24</td>
<td>6.00</td>
<td>5.82</td>
<td>5.64</td>
<td>6.72</td>
<td>0.00</td>
<td>12.00</td>
<td>0</td>
</tr>
<tr>
<td>Section_6</td>
<td>7.04</td>
<td>6.43</td>
<td>6.24</td>
<td>6.00</td>
<td>5.82</td>
<td>5.64</td>
<td>6.75</td>
<td>0.00</td>
<td>12.00</td>
<td>0</td>
</tr>
<tr>
<td>Section_7</td>
<td>7.03</td>
<td>6.42</td>
<td>6.24</td>
<td>6.00</td>
<td>5.82</td>
<td>5.64</td>
<td>6.83</td>
<td>0.00</td>
<td>12.00</td>
<td>0</td>
</tr>
<tr>
<td>Section_8</td>
<td>7.02</td>
<td>6.42</td>
<td>6.24</td>
<td>6.00</td>
<td>5.83</td>
<td>5.65</td>
<td>6.83</td>
<td>0.00</td>
<td>12.00</td>
<td>0</td>
</tr>
</tbody>
</table>

- **Gate_a**: 7.05, 6.42, 6.23, 5.98, 5.80, 5.61, 6.26, 4.60, 26.00, 0.05
- **Gate_b**: 7.05, 6.43, 6.24, 6.00, 5.81, 5.62, 6.51, 4.62, 26.00, 0.05
- **Gate_c**: 7.04, 6.43, 6.24, 6.00, 5.82, 5.63, 6.87, 4.65, 26.00, 0.05
- **Gate_d**: 7.03, 6.42, 6.24, 6.00, 5.82, 5.64, 6.84, 4.68, 12.00, 0.05
- **Gate_e**: 7.02, 6.42, 6.24, 6.00, 5.83, 5.65, 6.84, 4.69, 12.00, 0.05

Figure 5.4: Input to the probabilistic model of OpenEarth

The inputs of the model consist of return years, a corresponding matrix containing the water level and the crest level of each segment, the types of distribution of the variables, and the parameters of
the distributions. Moreover, some other parameters have to be assigned as well, e.g., probabilistic calculation method to apply, number of MCS sampling, number of bins for integral, failure mechanisms to be considered. Importantly, the limit state functions have to be specified. An example of the limit state function for overtopping/overflow is shown in Figure 5.4d. Some examples of the Matlab scripts of inputs are shown in Figure 5.4.

The outputs of the model are listed as follow:

- conditional failure probability given a return year
- unconditional failure probability per segment per failure mode
- unconditional failure probability of the sub-system

In addition, if FORM simulation is utilised, the model returns the following extra outputs:

- influence factors $\alpha$ of the variables in the limit state function
- design points for each failure mode and each segment

More importantly, since the scripts are editable, the outputs can be customised according to the demands and preferences.

5.3. Conditional Failure Probability

The conditional failure probability is listed in Table 5.1 and 5.2. In the tables, the water level with return periods of 1,000 years and 100 years were respectively adopted for example. 1,000 return year is the current safety standard for the flood defences at the lower reach of the Huangpu River, while 100 return year is the safety standard in the past. According to Mr. Chen, the 100-year water level was indeed reached in the history.

"NaN" means that the frequency of occurrence in the 500,000 Monte Carlo samples was 0, but the failure probability was not necessarily 0. To be precise, the failure probability was supposed to be smaller than 1/500,000. In the table, the overtopping/overflow probability of some segments reached 1.0, which were also not necessarily 1.0 given more Monte Carlo samples were taken.

Table 5.1: Annual conditional failure probability of the sub-system under 1000-year water level condition

<table>
<thead>
<tr>
<th>Segment</th>
<th>Overtopping/overflow</th>
<th>Piping</th>
<th>Structural failure</th>
<th>Closure failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section 1</td>
<td>1</td>
<td>2E-6</td>
<td>NaN</td>
<td>-</td>
</tr>
<tr>
<td>Section 2</td>
<td>1</td>
<td>2E-6</td>
<td>NaN</td>
<td>-</td>
</tr>
<tr>
<td>Section 3</td>
<td>0.99978</td>
<td>NaN</td>
<td>NaN</td>
<td>-</td>
</tr>
<tr>
<td>Section 4</td>
<td>0.999082</td>
<td>2E-6</td>
<td>NaN</td>
<td>-</td>
</tr>
<tr>
<td>Section 5</td>
<td>0.99670</td>
<td>0.3185</td>
<td>NaN</td>
<td>-</td>
</tr>
<tr>
<td>Section 6</td>
<td>0.999400</td>
<td>0.3185</td>
<td>NaN</td>
<td>-</td>
</tr>
<tr>
<td>Section 7</td>
<td>0.998538</td>
<td>0.3164</td>
<td>NaN</td>
<td>-</td>
</tr>
<tr>
<td>Section 8</td>
<td>0.998490</td>
<td>0.3160</td>
<td>NaN</td>
<td>-</td>
</tr>
<tr>
<td>Gate a</td>
<td>1</td>
<td>2E-6</td>
<td>NaN</td>
<td>0.01</td>
</tr>
<tr>
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<td>2E-6</td>
<td>NaN</td>
<td>0.01</td>
</tr>
<tr>
<td>Gate c</td>
<td>0.997710</td>
<td>2E-6</td>
<td>NaN</td>
<td>0.01</td>
</tr>
<tr>
<td>Gate d</td>
<td>0.998384</td>
<td>0.3149</td>
<td>NaN</td>
<td>0.01</td>
</tr>
<tr>
<td>Gate e</td>
<td>0.998138</td>
<td>0.3186</td>
<td>NaN</td>
<td>0.01</td>
</tr>
</tbody>
</table>
According to the two tables, overtopping/overflow is the most influential failure mode, and its conditional probability is much higher than expected. Especially, the floodwall section 1 and 2 with an estimated probability of nearly 1.0 is bound to fail even in the 100-year water level condition. More discussions and explanations about the high overtopping/overflow probability are presented in Section 6.3.

Piping is the second most important failure modes in the 100-year and 1000-year water level conditions. However, it is only significant in the floodwall section 5-8 and Gate d-e. It is further discussed in Section 6.4.

Failure probability due to failure of closure of the floodgates is less influential. However, compared with other failure modes, it was assumed to own the same conditional failure probability of 0.01 given any water level beyond the warning water level. So, the unconditional failure probability of it will be also high once lower water level conditions are incorporated.

Piping in Section 1-4 and Gate a-c, and structural failure for the entire sub-system are negligible.

### 5.4. Unconditional Failure Probability

Compared with the conditional failure probability, the unconditional failure probability reveals the actual safety level of the targets. They were obtained by integrate conditional failure probabilities in all the water level conditions. The following formulas were applied:

\[
P_F = \sum_{i=1}^{n} P(f \mid z_{0i}) \cdot P(z_{0i})
\]

(5.1)

For continuous probability functions, it becomes

\[
P_F = \int_{0}^{\infty} P(f \mid z_{0})f(z_{0})dz_{0}
\]

(5.2)

in which

- \(z_{0i} = \text{water level } (m + \text{MSL})\)
- \(P_{fi} = \text{unconditional failure probability when water level is } z_{0i}\)
5. System Reliability Analysis

\[ P(f \mid z_{0t}) = \text{conditional failure probability given a water level of } z_{0t} \]
\[ P(z_{0t}) = \text{probability of occurrence of } z_{0t} \]

It is the unconditional failure probability that reflects the true reliability of the flood defences. The background of deriving it is introduced in Appendix B.1. In the following subsections, the approach to calculating unconditional failure probability of the sub-system is introduced.

5.4.1. Fragility Curve

As is explained, failure could occur in any water level conditions despite the failure probability differs. Therefore, the concept of fragility curve is introduced, and linked to Formula 5.2, it represents the term \( P(f \mid z_0) \). Figure 5.5 presents the fragility curves and the exceedance probability of Gate a for example. The fragility curves indicate the conditional probability of various failure mechanisms.

In this example, Figure 5.5 shows that overtopping/overflow is the most influential failure mode for the gate a. Besides, overtopping/overflow would start at about \( 4m + \text{MSL} \) (1-year return period), and it would turn significant since \( 5m + \text{MSL} \) (10-year return period). It is explained in Section 6.2.3 that the overestimated wave height and underestimated critical discharge are the main reasons.

![Figure 5.5: Example of conditional failure probability (fragility curves) and exceedance probability with the respect of water level (Gate a)](image)

5.4.2. Convolution Integral

The second step is to implement Formula 5.2 provided that the probability distribution of the extreme water level is known. In order to perform an integral, Matlab was only able to execute Formula 5.1. Figure 5.6 gives an example. In the figure, "n" bins were taken, the area of each bin represents the probability of occurrence of a range of water level, which is the term \( P(z_{0t}) \) in Formula 5.2. Therefore,
5.4. Unconditional Failure Probability

the product of each area and the conditional probability at that water level represents the unconditional probability given a water level. By summing up all the products, the unconditional probability of one failure mode for one segment could be obtained.

In addition, the results could be more accurate if larger number of bins were taken. Thus, here comes a trade-off between accuracy of the results and the time consumed for computation. If faster computation was required, less accuracy had to be accepted as a compromise. In this study, the results were obtained by taking 1000 bins. It implies a bin width of 5\( \text{mm} \) for each range of water level.

The model was applied to every failure mechanism and every segment of the sub-system. The results are listed in Table 5.3.

![Figure 5.6: Visualisation of Formula 5.1](image)

Table 5.3: Annual unconditional failure probability of the sub-system

<table>
<thead>
<tr>
<th>Segment</th>
<th>Overtopping/overflow</th>
<th>Piping</th>
<th>Structural failure</th>
<th>Closure failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section 1</td>
<td>3.1E-01</td>
<td>4.3E-08</td>
<td>8.85E-06</td>
<td>-</td>
</tr>
<tr>
<td>Section 2</td>
<td>3.0E-01</td>
<td>4.3E-08</td>
<td>8.83E-06</td>
<td>-</td>
</tr>
<tr>
<td>Section 3</td>
<td>1.5E-01</td>
<td>5.3E-08</td>
<td>8.76E-06</td>
<td>-</td>
</tr>
<tr>
<td>Section 4</td>
<td>5.7E-02</td>
<td>3.9E-08</td>
<td>8.78E-06</td>
<td>-</td>
</tr>
<tr>
<td>Section 5</td>
<td>7.4E-02</td>
<td>3.1E-03</td>
<td>8.74E-06</td>
<td>-</td>
</tr>
<tr>
<td>Section 6</td>
<td>6.7E-02</td>
<td>3.1E-03</td>
<td>8.78E-06</td>
<td>-</td>
</tr>
<tr>
<td>Section 7</td>
<td>5.1E-02</td>
<td>3.1E-03</td>
<td>8.77E-06</td>
<td>-</td>
</tr>
<tr>
<td>Section 8</td>
<td>5.1E-02</td>
<td>3.1E-03</td>
<td>8.77E-06</td>
<td>-</td>
</tr>
<tr>
<td>Gate a</td>
<td>2.9E-01</td>
<td>4.9E-08</td>
<td>NaN</td>
<td>7.12E-03</td>
</tr>
<tr>
<td>Gate b</td>
<td>1.5E-01</td>
<td>4.9E-08</td>
<td>NaN</td>
<td>7.12E-03</td>
</tr>
<tr>
<td>Gate c</td>
<td>4.5E-02</td>
<td>4.5E-08</td>
<td>NaN</td>
<td>6.09E-03</td>
</tr>
<tr>
<td>Gate d</td>
<td>4.9E-02</td>
<td>3.2E-03</td>
<td>NaN</td>
<td>6.09E-03</td>
</tr>
<tr>
<td>Gate e</td>
<td>4.9E-02</td>
<td>3.1E-03</td>
<td>NaN</td>
<td>6.09E-03</td>
</tr>
</tbody>
</table>
5.5. Failure Probability of the Sub-system

It is worth mentioning the approach to combining failure probability per segment per failure mechanism listed in Table 5.3 into the reliability of the sub-system.

Generally, the components (i.e., segments plus failure modes) constitute a serial system in which the system would fail once one or more components failed. Dependency between one another would influence the results of combination. Regarding correlation, a number of methods could be applied for estimation such as Hohenbichler [25] method and Ditlevsen method [26]. In this loop, correlation was not dealt with. Instead, the elementary lower and upper bounds were simply taken.

5.5.1. Sequential Order of Combination

In order to combine the failure probability of components. There are usually two steps to take: one is to combine the failure probability with the respect of failure mechanisms; the other is to combine the failure probability with the respect of flood defence segments.

Either of the two steps can be done prior, while the other should be performed afterwards. The two sequential orders would lead to distinct results, because the correlation of the segments and that of the failure modes are different. However, in the first loop of the study where correlation was not involved, the former step was taken firstly and the latter secondly. The steps are visualised in Figure 5.7

![Figure 5.7: Process of combination](image)

5.5.2. Overflow/overtopping

Overtopping/overflow is usually considered to be highly spatially correlated. Water levels and crest levels are usually the predominant factors that affect the reliability of segments against overtopping/overflow. They are usually correlated to a large extent.

Therefore, for a simple approximation, a lower bound of \( \max(P_f) \) was taken. However, in reality, the dependency among segments concerning overtopping and overflow might not be that high, so failure probability of it should be slightly higher than the lower bound.
5.5.3. Piping

On the contrary, in terms of piping, length effects make great contributions. Piping failure was mainly related to soil type, which could vary significantly along the distance. Moreover, the soil property was not correlated one another along the stretch. As a consequence, the upper bound of $P_f = \Sigma R_i$ was applied to a mutually exclusive case.

5.5.4. Structural Failure

The correlation with the respect of structural failure highly depends on the types of the structures and their profiles. For instance, from Section 1 to Section 4, the profiles are homogeneous, so the correlation among them should be very high. In contrast, the correlation between Section 1 and Section 8 is very low, because they have completely different profiles.

However, assumptions were made that only two types of floodwall profiles were adopted along the sub-system. In reality, the number of types of the segments must be more than two. Conservatively, the upper bound of $P_f = \Sigma R_i$ was applied for this failure mode.

5.5.5. Failure of Floodgates' Closure

Dependency among closing regimes of different floodgates could affect the failure probability of the system. Take the five hypothetic floodgates in the stretch for example. Each of them has a 0.007 unconditional probability of failure closure: $P_1 = P_2 = P_3 = P_4 = P_5 = 0.007$.

In an extreme case, the five floodwalls might be operated by a single group. Operators were trained in the same way and the closing regimes of the gates were also exactly the same. Then a plausible assumption is that the closing process of these five gates were fully dependent. If one gate is shut successful, the other gates can be shut as well. As a consequence, the failure probability for all the floodgates within the stretch $P_f$ is equal to $\max(P_f) = 0.007$.

On the other hand, there exists another extreme situation. The closure of those gates are fully exclusive. It means if one of the floodgates failed, the remaining gates is not possible to fail during the same flood. For instance, if a gate failed, water flowed into the backland. The influx consequently decreased in front of other gates, and operators could absolutely manage to shut the gates. In this case, $P_f = \Sigma P_i = 0.007 \times 5 = 0.035$.

Another special situation is that procedures of closing gates are independent. For instance, the gates and operators were affiliated to diverse groups. Closure of each gate could be considered as an independent procedure. In this case, $P_f = 1 - \Pi(1 - P_i) = 0.0345$.

Actually, in most situations, closing procedures are somewhat dependent on one another. Their degree of dependency influences the $P_f$. Taking the upper limit and the lower limit, $P_f$ ranges from 0.007 to 0.035.

5.5.6. Combination of Failure Modes

In order to combine the failure probability of all the failure modes into that of the system, the same method was taken, and both the upper and lower bounds were taken. It turned out to be a system failure probability ranging from $\max(P_f)$ to $\Sigma(P_f)$.
5.6. Results of Sub-system Reliability

Table 5.4 shows the results of the failure probability combination. The total failure probability is about 0.31-0.36 per year. Again, it can be seen from the table that overtopping/overflow is estimated to be the most important failure mechanism.

Piping and closure failure of the floodgates are also important due to length effect. Provided that more floodgates or more than 10km stretches were incorporated into the system, the length effect would lead to a higher significance of piping and closure failure.

Specific discussions about the results are presented in Chapter 6.

Table 5.4: Unconditional failure probability of the sub-system

<table>
<thead>
<tr>
<th>Segment</th>
<th>Overtopping</th>
<th>Piping</th>
<th>Structural failure</th>
<th>Closure failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section 1</td>
<td>3.1E-01</td>
<td>4.3E-08</td>
<td>8.85E-06</td>
<td>-</td>
</tr>
<tr>
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<td>4.3E-08</td>
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<td>-</td>
</tr>
<tr>
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<td>1.5E-01</td>
<td>5.3E-08</td>
<td>8.76E-06</td>
<td>-</td>
</tr>
<tr>
<td>Section 4</td>
<td>5.7E-02</td>
<td>3.9E-08</td>
<td>8.78E-06</td>
<td>-</td>
</tr>
<tr>
<td>Section 5</td>
<td>7.4E-02</td>
<td>3.1E-03</td>
<td>8.74E-06</td>
<td>-</td>
</tr>
<tr>
<td>Section 6</td>
<td>6.7E-02</td>
<td>3.1E-03</td>
<td>8.78E-06</td>
<td>-</td>
</tr>
<tr>
<td>Section 7</td>
<td>5.1E-02</td>
<td>3.1E-03</td>
<td>8.77E-06</td>
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<td>5.1E-02</td>
<td>3.1E-03</td>
<td>8.77E-06</td>
<td>-</td>
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<td>Gate a</td>
<td>2.9E-01</td>
<td>4.9E-08</td>
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<tr>
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<td>3.2E-03</td>
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<tr>
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<td>7.0E-05</td>
<td>3.3E-02</td>
</tr>
</tbody>
</table>

Total: 0.31-0.36
Discussions and Suggestions

After taking the procedures introduced in the preceding chapters, the individual failure probability per failure mechanism per segment and failure probability of the sub-system were both derived. Some findings are presented in this chapter. Moreover, some results are not consistent with the expectations, so the differences between the results and the expectations are discussed in this chapter. In the final section, a summary of recommendations is given for the phase II of the study.

6.1. Comparison of Models

In order to perform a reliability analysis, a tailor-made model was set up, which has been introduced in Section 5.2. Herewith, a comparison with two other alternatives, i.e., PC-Ring and Prob2B is presented.

PC-Ring is a relatively advanced software, developed particularly for the VNK2 project in the Netherlands. Specific models were embedded in the PC-Ring to calculate the reliability of different flood defences: dikes, dunes, floodwalls, floodgates, etc. The failure mechanisms in PC-Ring are also various, including overflow/overtopping, uplift and piping, structural failure, erosion of dunes, sliding of slopes, closure failure of floodgates, etc. Various simulating methods are available in PC-Ring. Besides, the PC-Ring software is much more versatile than the tailor-made method in this study and Prob2B, although it also has inevitable drawbacks. For the details about PC-Ring, the manuals [24][27][28] can be referred to.

Prob2B is a simple software for probabilistic calculation. Like the tailor-made model, to apply it to the flood defence system along the Huangpu River, the limit state functions have to be specified, and reliability calculation in bulk is not possible in Prob2B.

The table 6.1 and 6.2 show the comparison among PC-Ring, Prob2B, and the tailor-made model. To summarise, PC-Ring is the most comprehensive and developed software in which numerous factors are taken into consideration such as spatial correlation and time correlation. However, a key drawback of it is the high requirement of data preparation. Prob2B (free version) is the simplest tool among the three, the straightforward interface ensures a quick implementation of the probabilistic calculation. Nevertheless, the software can only calculate the probability of one failure mode for one segment each time. Poor expansibility makes it difficult to proceed a group calculation by additional programming.
Table 6.1: Comparison of PC-Ring, Prob2B, and tailor-made model

<table>
<thead>
<tr>
<th>Limit state function</th>
<th>PC-Ring</th>
<th>Prob2B (free version)</th>
<th>Tailor-made model</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pre-defined</td>
<td>- Customized</td>
<td>- Customized</td>
</tr>
<tr>
<td>Input</td>
<td>Organised dataset</td>
<td>- Single number, no dataset</td>
<td>- Less organised dataset</td>
</tr>
<tr>
<td></td>
<td>Manually input parameters of distributions</td>
<td>- Manually input parameters of distributions</td>
<td>- Manually input parameters of distributions</td>
</tr>
<tr>
<td></td>
<td>Spatial correlation and time correlation</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Calculation method</td>
<td>MCS</td>
<td>- MCS</td>
<td>- MCS</td>
</tr>
<tr>
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<td>FORM</td>
<td>- FORM</td>
<td></td>
</tr>
<tr>
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<td>SORM</td>
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<td></td>
</tr>
<tr>
<td></td>
<td>DS</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Output</td>
<td>Conditional $P_f$ of components</td>
<td>- Conditional $P_f$ of components</td>
<td>- Conditional $P_f$ of components</td>
</tr>
<tr>
<td></td>
<td>Unconditional $P_f$ of components</td>
<td>- Design point</td>
<td>- Unconditional $P_f$ of components</td>
</tr>
<tr>
<td></td>
<td>Design point</td>
<td>- Influence factor</td>
<td>- Design point</td>
</tr>
<tr>
<td></td>
<td>Influence factor</td>
<td></td>
<td>- Influence factor</td>
</tr>
<tr>
<td></td>
<td>Unconditional $P_f$ of system</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 6.2: Pros and cons of PC-Ring, Prob2B, and tailor-made model

<table>
<thead>
<tr>
<th>Advantage</th>
<th>PC-Ring</th>
<th>Prob2B (free version)</th>
<th>Tailor-made model</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Comprehensive and organised</td>
<td>- Easy to use for a single case</td>
<td>- Highly customizable in Matlab</td>
</tr>
<tr>
<td></td>
<td>Include much more factors</td>
<td>- Straightforward interface</td>
<td>- low requirement of data quality</td>
</tr>
<tr>
<td></td>
<td>Easy to use given good dataset</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Disadvantage</td>
<td>High requirement of data quality</td>
<td>- Limited functions in free version</td>
<td>- Less organised</td>
</tr>
<tr>
<td></td>
<td>Difficult to customise limit state functions and calculation steps</td>
<td>- Strict requirement of computational environment (Matlab 2009 or lower is required)</td>
<td>- Bulky calculation due to code redundancy</td>
</tr>
<tr>
<td></td>
<td>Unconditional $P_f$ of components</td>
<td>- Unable to calculate in bulk</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Design point</td>
<td>- Poor expansibility</td>
<td></td>
</tr>
</tbody>
</table>

In a proof of concept method, the quality of input data does not fulfill the requirement of PC-Ring. Also considering that the accuracy is not the prior issue in this phase, the tailor-made model was therefore the better option. The customizability makes it possible to fulfill the academic demands, for instance, to perform a sensitivity analysis. However, the calculation rate could be slower than the other two, and a great amount of computational resources were occupied owing to the code redundancy.

6.2. Findings about the Sub-system

6.2.1. High Failure Probability of the Sub-system

The concluded failure probability of the sub-system is about 0.31-0.36 per year. Take a look at the failure probability in a time span of multiple years instead of one year. Assume that the defence system could remain for 10 years. The failure probability of at least once 10 years can be derived as follow:

$$P_{f,10} = 1 - (1 - P_{f,1})^{10} > 0.98$$  \hspace{1cm} (6.1)

in which:

- $P_{f,10}$ = unconditional failure probability per year
- $P_{f,1}$ = unconditional failure probability every 10 years

In other words, within every 10 years, the failure of the selected stretch is nearly bound to happen at least once.

From another angle, take the whole flood defence system along the Huangpu River into account. The entire floodwall is 46 times longer than the length of the stretch, and the number of floodgates are 300 times larger than the 5. The selected stretch is in a serial system with other stretches. It is imaginable that the flooding frequency would be at least once a year in the whole system, which is apparently contrary to the observations. Therefore, the yearly failure probability concluded in the first loop was a bit overestimated.
6.2. Findings about the Sub-system

6.2.2. Decisive Failure Modes and Weak Points

By comparing probability of failure modes, overtopping/overflow is the most contributing failure mode (86% in Table 6.3). Floodgates’ closure failure has less influence, and piping is even less. Under the first assumption, structural failure is unlikely to occur.

Regarding overflow/overtopping, Section 1, Section 2, Gate a are the decisive segments in the sub-system, as the overtopping/overflow probability of the sub-system depended on the segments with maximal failure probability. In the failure mode of piping, the weaker segments are floodwall Section 5-8, Floodgate a and Floodgate b. Therefore, the profile B, the representative profile type for them, is more vulnerable to piping.

Therefore, overtopping/overflow of Section 1, Section 2, and Gate a are the decisive components in the system. In fact, these three segments are located at the part of the stretch with lower crest levels, which can be seen from Figure 2.9. In order to reduce overestimation of the flooding hazard, these components are focussed on and further discussed in the following sections.

Table 6.3: Contributions of components to the unconditional failure probability of the sub-system

<table>
<thead>
<tr>
<th>Segment</th>
<th>Overtopping</th>
<th>Piping</th>
<th>Structural failure</th>
<th>Closure failure</th>
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<tbody>
<tr>
<td>Section 1</td>
<td>3.1E-01</td>
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<td>Section 4</td>
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<td>4.5E-02</td>
<td>4.5E-08</td>
<td>NaN</td>
<td>6.09E-03</td>
</tr>
<tr>
<td>Gate d</td>
<td>4.9E-02</td>
<td>3.2E-03</td>
<td>NaN</td>
<td>6.09E-03</td>
</tr>
<tr>
<td>Gate e</td>
<td>4.9E-02</td>
<td>3.1E-03</td>
<td>NaN</td>
<td>6.09E-03</td>
</tr>
<tr>
<td>Combined</td>
<td>3.1E-01</td>
<td>1.9E-02</td>
<td>7.0E-05</td>
<td>3.3E-02</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td>0.31-0.36</td>
<td></td>
</tr>
</tbody>
</table>

6.2.3. Design Water Level

In the probabilistic calculation of each component, it is possible to derive the design water level at which the highest failure probability appeared. This goal was achieved by simply finding out the greatest product \( P(f | z_0) \cdot P(z_0) \) and its corresponding water level \( z_0 \).

As is shown in Figure 6.1, the design point of the water level was 4.97\( m \) + MSL. It implies that most possibly, the failure of the sub-system could occur under the condition of 4.97-meter water level, which corresponds to the 10-year return period.

Mathematically, the design point of the water level came out to be 4.97\( m \) + MSL possibly because probability density of water level plays the dominant role instead of conditional failure probability in the convolution integral. The conditional failure probability in the range of intermediate and high water level (more than 4.5\( m \)) are universally high. In contrast, the probability density at the intermediate water level
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(around red dash line in Figure 6.1) is much higher than that at the higher or lower water level, so the design point was found at the intermediate range (around 5m).

Figure 6.1: Design point of the water level for the sub-system

Figure 6.2: Unconditional failure probability (fragility curves) and exceedance probability of water level (gate a)
From physical perspective, the design point is highly dependent on the overtopping/overflow of the section 1, Section 2 and Floodgate a, which were identified as the decisive components in the subsystem. Additionally, the fragility curve for Gate a (Figure 6.2) offers consistent findings: the failure due to overtopping/overflow starts to show up at about 4m + MSL (1-year return period), and turns significant since an intermediate level of around 5m + MSL (10-year return period).

At the floodgate a, the freeboard height is about 1.3m, given a water level of 4.9m. The crest height of the floodgate is high enough against water without the presence of wave. However, the wave height was estimated to be up to 0.7m in the first loop. 4.9m still water plus 0.7-meter-high wave leads to a high dynamic water level, close to the crest level. Moreover, in front of a vertical structure with a submerged toe, waves can break at the structure [29] and run up and over the structure. Demonstrations can be seen from Figure 6.3. This mechanism poses threat to the vertical flood defences in spite of a water level lower than the crest level.

![Figure 6.3: Waves' impact on vertical structure (Pullen, T. et al, 2007)](image)

### 6.3. High Probability of Overflow/overtopping

As is shown in Table 5.1 and 5.2, the failure probability of some segments by overflow or overtopping is extraordinarily high (up to 99%) under the condition of 1,000-year return period. In the condition of 100-year return period, the overtopping/overflow probability is also high (77% - 99%).

According to the past events along the Huangpu River, overtopping/overflow could occur, but the failure probability was of no chance to be that high. In the history, the most severe situation ever was that the water level approached the 200-year water level condition. In this event, several sections of floodwalls at the upper reach failed due to overflow. However, there was no reports mentioning that any of the sections in the selected stretch failed by overtopping or overflow. Therefore, the overtopping/overflow probability is not in line with the expectations.

One source of the exaggeration could be the conservative assumptions of the variables. Influence factors (\(\alpha\) provided by FORM calculation can show the importance of the variables in the limit state functions. In other words, those more important variables should be paid attention to in this section. Take the floodwall section 8 for example. Table 6.4 presents influence factors in the overtopping/overflow calculation.

#### Table 6.4: Influence factor: overflow/overtopping (Section 8, 1,000 return years)

<table>
<thead>
<tr>
<th>Variable</th>
<th>(\Delta l)</th>
<th>(F)</th>
<th>(U)</th>
<th>(d)</th>
<th>(z_0)</th>
<th>(z_c)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\alpha)</td>
<td>-0.0399</td>
<td>-0.4633</td>
<td>-0.7869</td>
<td>-0.0024</td>
<td>-0.2099</td>
<td>0.3472</td>
</tr>
</tbody>
</table>

From the table, it is concluded that wind speed and fetch length rank the first and second most influential variables, followed by crest level and water level. However, crest level and water level are determined...
with relatively reliable data. In other words, despite influence factors of them are high, they are relatively certain and are not the primary causes of high overtopping/overflow probability.

In addition, another essential value, critical discharge \( q_c \), should be also taken into account. In the calculation, \( q_c \) is considered as a constant whose value is determined according to the reference case in New Orleans. In reality, critical discharge in a case of defensive structures is not sufficient. There is no universally applicable standard about effects of discharge on floodwalls or floodgates. Therefore, it is crucial to find out how structures would behave and how water would spread behind the structures given overtopping/overflow.

In the following subsections, sensitivity analysis is performed. The approach is to change One-Factor-At-a-Time (OFAT) and to observe the influences of above-mentioned variables. The Section 1 and 8 are taken for example, as Section 1 was considered potentially under the threats of overflow while Section 8 is considered to be more likely to encounter overtopping. 1,000 return year and 100 return year were chosen as water level conditions.

### 6.3.1. Critical Discharge

Take Section 1 and 8 for example. The line chart 6.4 indicates the variation of failure probability by overtopping or overflow versus increasing critical discharge. From the graph, multiple findings can be pointed out.

![Sensitivity analysis: critical discharge](image)

**Figure 6.4: Sensitivity analysis: critical discharge.** Except critical discharge, other variables are fixed. Red dash-dotted line represents the assumption made originally; the curves represent different combinations of water level and crest level.

Firstly, by observation, the initial assumption for critical discharge of 10\( l/m/s \) leads to a conditional 80%-100% failure probability \( (P_f) \). It is obviously not possible in practice. If an expected conditional failure probability is set to 0.1, then at least a \( q_c \) of 100\( l/m/s \) should be assumed, while in the case of Section 1 with 1,000 return years, 3,000\( l/m/s \) is required. If so, the damage brought by the 3,000\( l/m/s \) has to be estimated. Note that, the backland behind the structures are also covered by robust concrete, so the strength is rather high.

Moreover, compared with failure probability of Section 8, that of the section 1 has a much higher failure chance. It makes sense, because according to the crest level shown in Figure 2.8, there is a remarkably low-level section between 19.5\( km \) and 21\( km \). By contrast, such a change is not present for the water...
levels at this section. Therefore, it is logical to see such a difference of failure probability between the two sections.

Thirdly, the curve of Section 1 (1,000 years) indicates a long straight line at 1.0, and it starts to drop from almost 1,000l/m/s. The reason can be found from Figure 2.8. Under the water level condition of 1,000-year return period, the water level exceeds the crest level in Section 1. In this case, overflow occurs and Formula 3.3 is activated. The extra term that represents water flow directly attacking backland results in the huge failure probability in the section 1.

Once Formula 3.3 was activated, a phenomenon can be observed. In order to show this, a simple deterministic calculation was executed, and mean values of the variables at the section 1 (1,000 years) were taken only for an approximation. The results show that contribution of the wave term, $\sqrt{g(H)}$, is 8 times larger than that of the water flow term, $0.55\sqrt{-g(z_c - \Delta l - z_o)}$. Effects of waves are much larger than that of water level, which is counterintuitive. A possible explanation could be related to the overestimation of wave height owing to the overestimation of the wind properties.

### 6.3.2. Wind Speed

The same approach was taken again for wind speed that is the most influential variable according to the influence factor ($\alpha$). Figure 6.5 shows how failure probability drops with decreasing wind speed. Standard deviation is set as a constant of 5m/s in each condition.

![Figure 6.5: Sensitivity analysis: wind speed. The standard deviation is set as a constant of 5m/s in each condition, and other variables are fixed.](image)

As is shown in the graph, firstly, failure probability of Section 1 (1,000 years) doesn’t reduce much, staying around 1.0, even though the mean value is reduced to 0. Section 1 is the segment with insufficient crest level in the condition of 1,000 return years. In this case, overflow occurs without waves’ effects. Hence, in the overflow-dominated sections, accuracy of wind speed doesn’t influence much.

By contrast, wind speed does make great efforts in the other three case. When it reaches 10m/s, the average wind speed of typhoon in Shanghai, the failure probability of these three cases drops below 0.2, which is more in line with the expectations.
6.3.3. Fetch Length

In Figure 6.6, the similar procedure is taken for fetch. By shortening the fetch length, failure probability falls. However, the intensity of the decline is smaller than that by decreasing wind speed. Therefore, fetch is less influential than wind speed in the limit state function of overflow/overtopping.

Here is an interesting phenomenon that, when fetch was shortened below 500m, a remarkable rise showed up. It was apparently unusual. When fetch is small enough, standard deviation of 500m plays a role. Consequently, fetch length could be sampled negative in the Monte Carlo simulation, which was not possible in the reality. Basically, the root is the normal distribution that was assumed for fetch. Normal distribution, in other words, allows negative value. This issue could also occur in the analysis of other variables, such as wind speed. A solution could be to use, for example, lognormal distribution instead of normal distribution. Of course, if in reality, standard deviations were not remarkable, negative values would hardly show up or would not have great effects on the results.

![Figure 6.6: Sensitivity analysis: fetch.](image)

The standard deviation is set as a constant of 500m in each condition, and other variables are fixed.

6.3.4. Wind Direction

Since wind properties are rather essential in the limit state functions for overtopping and overflow. It is logical to speculate that wind direction also plays an important role in the overestimation. Initially, the wind speed was simplified to be perpendicular to the structures’ orientations. By simple assumption, if the occurrence probability of wind among 16 directions (0°, 22.5°, ..., 337.5°) is equal, then a decrease of the failure probability by a factor of 10 is rather influential to the results.

6.3.5. Effects of Wave Height and Crest Height

Take a close look at the effects of wave. Deterministic calculations were taken by varying the magnitude of wave height, and the results are shown in Figure 6.7. The level differences between crest and water, \( R = z_c - \Delta l - z_a \) is defined as Crest height or freeboard. A consistent finding is that the discharge is enormous in the 10,000-year condition for Section 8 and the 10,000-year and the 1,000-year condition for Section 1, because overflow takes place. In these situations, even if wind effects were not involved (H=0), the discharge still remains high.
In the case of overtopping, for example, 1,000-year condition in Section 8, discharge increase tremendously with rising $H$. In comparison, the increasing tendency is much less significant for the curve of 10,000-year in the same section, which is claimed to be overflow situation. Therefore, in the latter condition, wave height does affect discharge, but the influences are limited.

![Figure 6.7: Overtopping/overflow discharge in the 10,000-year, 1,000-year, 500-year, and 100-year water level condition versus increasing wave height. The red dash-dotted lines represent the wave height condition applied in the first loop. Crest height $R = z_c - \Delta l - x_0$, and negative $R$ indicates the water level exceeds the crest level.](image)

![Figure 6.8: Overtopping/overflow discharge versus increasing crest height $R$. Crest height $R = z_c - \Delta l - x_0$, and negative $R$ indicates the water level exceeds the crest level.](image)

Besides, as is demonstrated in Figure 6.8, overflow/overtopping discharge reduces with increasing crest height. The three lines are divided into two parts at the crest level 0, in essence, overflow and
6. Discussions and Suggestions

In the two parts, lines have distinct dropping rates. It is consistent with the conclusion made before that, when overflow occurred, the discharge would remain high, but in the domain of overtopping, increasing crest height could significantly reduce the discharge.

In conclusion, to mitigate the overflow hazard, it is a more effective measure to raise the crest level to exceed the water level than wave dissipation. In the case of overtopping, wave dissipation becomes important.

In the loops of reliability analysis, the priority should be to calibrate wind speed for the segments encountering overtopping. In the segments with low crest levels, it is recommended to concentrate on other factors, for instance critical discharge, wind direction, and fetch.

6.3.6. Closure

In this loop, waves have drastic impacts on reliability of the flood defences. Variables that influence wave height, including wind speed, wind direction, fetch length, make great contributions to the high overflow/overtopping probability. In the first phase, very conservative assumptions were made that wave height \( H = 0.7 \text{m} \). Hence, accuracy of these variables are supposed to be improved, and it is also interesting to estimate the correlation between extreme wind speed and extreme water level.

Moreover, possible causes of high failure probability could be other factors: the direction of typhoon and the threshold of failure (critical discharge).

In the initial assumption, effects of wind direction was ignored, but in fact wind could come from any direction in consideration of the rotating features of typhoon. However, in Shanghai, typhoon mainly comes from the southeast. Despite typhoon rotates, the main direction might still be from the southeast. Then the direction of the wind-induced waves should be coupled with wind directions. So, waves would not affect significantly on the stretch at the east bank. In this case, wave effect would affect the opposite side (west bank) of the stretch. Moreover, there might be reflection effects of the incident waves, because vertical walls could significantly reflect waves. To some extent, standing wave might show up. It is still unknown how much effects it could be.

The critical discharge for hydraulic structures are also interesting to study on, as there has not been any authoritative research that presents this threshold in Shanghai.

In addition, note that in Section 1, waves are not the main factors to cause failure. Instead, overflow takes the responsibility, and the probability of failure is rather high. Within this section, there might be some other hydraulic structures or measures that can help withstand flooding. These factors are unknown, and can be interesting to investigate.

Finally, another alternative to cause such a massive overtopping hazard is that the limit state functions applied in the VNK project is not applicable in Shanghai’s case. The model factors should be calibrated, or even a new limit state function should be developed.

6.4. Probability of Piping

In the first loop calculation, piping probability under the water level condition of 1,000 return years is up to 0.32 per section in the case of profile B. Considering length effects for piping, the piping probability for the entire stretch could reach 1.0, which is also an overestimation.

By observing \( \alpha \) values in the Table 6.5, importance of "C" factor ranks the first, followed by seepage
6.4. Probability of Piping

length. On the contrary, inner and outer water levels which are usually considered rather essential are not influential in this case.

Table 6.5: Table of influence factor: piping (Section 8, 1,000 return years)

<table>
<thead>
<tr>
<th>Variable</th>
<th>$z_0$</th>
<th>$z_p$</th>
<th>$C$</th>
<th>$L$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$a$</td>
<td>-0.0764</td>
<td>0.1527</td>
<td>-0.7978</td>
<td>0.5782</td>
</tr>
</tbody>
</table>

Data of water level are relatively more reliable, while many assumptions were made for the other three variables. Therefore, in this section, the effects of the three variables are discussed.

6.4.1. Ground Water Level

In this study, a logical assumption was made that the ground water level was equal to the average water level of the Huangpu River due to slow reaction in clay layers. A sensitivity analysis is shown in Figure 6.9. Failure probability is negatively correlated with ground water level.

In the first loop, average water level during summer was taken to represent ground water level. In reality, the ground water should be higher than it. According to the graph, when the ground water level exceeds $3m + MSL$, failure probability decreases slightly. It proves that the ground water level is not a very influential variable.

Figure 6.9: Sensitivity analysis: ground water level (Section 8, 1,000 return years)

6.4.2. Seepage Length

Seepage length depends on the arrangement of the structures and soil properties. Two types of structural profiles were assumed to represent the entire 10km stretch. In the section 1 (Profile A), long sheet piles were adopted. They guaranteed the safety against piping. Therefore, the sensitivity analysis only aims at Profile B, and it is demonstrated in Figure 6.10.

According to Figure 2.4b, the assumed seepage path with a length of 12m is along the interface between structure and soil. It ends at the top of permeable filter layer. In reality, water could flow through weak type of soil such as sand in the clay layer. The seepage length can be reduced to 5m - 6m. From Figure 6.10, if the seepage length was 6m, failure probability could exceed 0.9. A logical assumption
is that seepage length could vary from 6\text{m} to 12\text{m}, in accordance with the actual condition. Within this range, ground water level has a remarkable effect on failure probability.

![Figure 6.10: Sensitivity analysis: seepage length](image)

### 6.4.3. "C" Coefficient

"C" coefficient was quoted from China’s official stipulation for flood defence design [15]. It depends on local soil types and whether filter layer is present. Generally, it ranges from 2-4 for clay layer. In Figure 6.11, failure probability versus "C" ranging from 2 to 4 is illustrated. The figure shows that, given $C = 4$, failure probability can be 3 times larger in 100-year condition and 5 times larger in 1,000-year condition than that with initial assumption ($C = 3$).

![Figure 6.11: Sensitivity analysis: "C"](image)

### 6.4.4. Discussion

Generally speaking, several conclusions can be made. Seepage length and "C" coefficient are relatively more influential. A small change of their mean values could cause a significant failure probability variation. By approximation, the error could reach a factor of 10, if data were not reliable.
Moreover, the applicability of the limit station function, Formula 3.5, is doubtful. The formula was built in the similar form to Bligh and Lane formulas. However, the Bligh and Lane are not applicable for the clay or silt. Actually, piping can hardly occur in clay which spreads widely under the ground by the Huangpu River. The formula was still applied in the design stipulation [15], because it is assumed that piping could occur in the sand layers mingled in the clay layers.

On the other hand, the inapplicability can be proved statistically. "C" is difficult to determine and is usually decided through expert judgement. However, the influence of "C" is extremely tremendous in the limit state function. So, great uncertainty and errors would stem from the expert judgement, and that is why Bligh and Lane are rarely recommended for probabilistic calculations, neither is Formula 3.5. Therefore, a more advanced formula is summoned with the assist of more detailed data.

6.5. Negligible Probability of Structural Failure

Overall instability is an important failure mechanisms for hydraulic structures in the Netherlands. Also, one of the floodwalls in New Orleans failed due to the instability. In contrast, it is usually not a problem for Shanghai in the circumstance of flooding. Along the Huangpu River, an extreme water level of 1/1,000 per year is only about 1.5\( \text{m} \) higher than the inner water level, which does not threat the concrete structure in a way of overall instability.

With conventional assumptions for the probabilistic calculation, structural failure of floodgates and floodwalls could hardly happen. On the contrary, in Shanghai, rupture of structures did happen from time to time during flooding. Instead of conventionally single event, the structural failure might be considered in a sequence of events (parallel system). It was also verified by Mr. Chen. S. He reported that ship collision and adjacent underground construction cause aging of the flood defences. Therefore, the scenarios of failure could be schematised as follow:

- **Ship Collision**
  - Ship collision → concrete/steel rupture → failure of reparation → structural failure by water pressure during floods

- **Underground Construction**
  - Uneven settlement → floodwall rupture → failure of reparation → structural failure by water pressure during floods

Each scenario is a parallel systems. Even though these scenarios can contribute to the failure of the system during flooding, the probability of occurrence is rather small. In the next stage, it is not prior to include them.

Last but not least, another possibility is that ship collision can happen right during flooding. In this case, massive collision might cause collapse of the structures. As a result, water directly flow onto the land, then damage occurs. It is worthwhile to include this scenario in the future study. Note that, calculated failure probability should contain the probability of occurrence of ship collision as a condition.

6.6. Threshold for Failure of Floodgates due to Non-closure

In the first loop, assumption were made that probability of closure failure is 1/100 per demand, and a shut is demanded once water level exceeded the warning water level. "Failure" was defined when the gates could not close. However, in the VNK2 project, closure failure is schematised as a series of events, which is demonstrated in Figure 5.3.
Considering the scenarios instead of the single event, another interesting topic could be included: how does water level condition affect probability of closure failure? For instance, the probability of closing process might fluctuate with different water level, because higher water level may exert more difficulty to the closure of gates. Besides, since it is usually tolerable to allow some amounts of water to flow onto the backland, the storage capacity behind the gates might also be a factor.

In addition, rather than the assumption of only 5 floodgates, about 20 floodgates could be present within the 10km stretch. Taking the upper bound of $P_f$ for the serial system, the $P_f$ could reach 0.2 \((= 0.01 \times 20 = 0.2)\). Hence, failure of closure is rather important in the reliability analysis and worth studying on in the further stages.

### 6.7. Recommendations

In this section, recommendations are provided for the next stages of the study. Among them, overtopping/overflow mechanism is prior to study on, as they contributed the most to the failure probability of the system.

- **Overflow/overtopping**
  - Analyse statistically extreme wind speed, coupled with wind direction and fetch. In order to do it, wind information is necessary and has already been collected.
  - Figure out the correlation between (extreme) wind speed and (extreme) water level. Hourly water level between 1949 and 2014 are needed.
  - If the wind waves mainly propagate away from the stretch, what would be the effects of wave reflection by vertical floodwalls at the west bank?
  - Verify applicability of overtopping/overflow limit state functions.
  - Define the critical discharge for the floodwalls and floodgates. Study on the waves’ and overflow’s impacts on the collapse of structures and on the expansion of water behind structures. These impacts might be associated with the duration of waves, backland roughness, and the amount of inflow.

- **Piping**
  - Study on why piping could occur in the soil condition, which seems to be majorly comprised of clay layer. Typical soil types and their distribution at the profile A and B are needed.
  - Study on how seepage could develop (seepage path) in the soil especially for the profile B with slope revetment and filter layer.
  - Adopt or develop a more informative and more reliable limit state function for piping.
  - Deal with the piping hazard at the junctions between floodwalls and floodgates. In order to do this, information about the on-site technical solution at the junctions is necessary.

- **Closure Failure**
  - Consider the storage capacity behind the floodgates. In this case, closure failure should be redefined into a series of events.
  - Study on the correlation of closure failure among the floodgates within the stretch.
  - How could different water level conditions and wind conditions affect the failure of closure.
• Others
  – Define the correlations of segments and failure modes and include them into the process of combination.
  – Approximate the probability of direct collision by ships on the structures during flooding.
  – Investigate why the weak points (i.e., Section 1, Section 2, Gate a) exist. For instance, are there any secondary flood defences on site, or is the consequence of flooding rather small?
  – Incorporate flooding scenarios and consequence analysis to perform a comprehensive flood risk analysis.
Phase II: Specific Analysis of Overtopping/overflow
As is concluded in the Chapter 6 of the first loop, overtopping/overflow was identified as the predominant failure mode, and wind wave contributes the most to it. In this chapter, wave-related factors are concentrated on, including wind speed, wind direction, effective fetch, and wind duration.

Specifically, in the first section, wind speed in typhoon events, as the most vital variable, is modelled. An extreme value analysis was performed to derive the extreme wind speed versus return periods. In the following sections, other wind properties, i.e., wind direction, fetch length, and wind duration are reported. The final section is discussions including the updated reliability of the sub-system with the updated wind properties. Additional, effects of typhoon on wind direction and wind duration are discussed as well in this section.

7.1. Extreme Wind Speed

In the precedent chapters, the importance of wave height has been already stated. By approximation, 0.7m wave height could lead to tremendous overtopping or overflow. However, in order to adopt the 0.7m wave height, a number of assumptions were made. For instance, the 100-year wind speed was adopted for all the conditions. Another assumption was that wind always attack the flood defence perpendicularly. These assumptions led to the overestimation of the threats of wind waves, and specific consideration was required in order to improve the accuracy of the results.

7.1.1. Dataset

Dr Wu, Z. et al. introduced a dataset in Statistical analysis of wind velocity and rainfall intensity joint probability distribution of Shanghai area in typhoon condition [30]. The dataset contains 10-minute maximum sustained wind speed in the 47 typhoon events from 1971 to 2007, which are listed in Table 7.1. The measuring point is located on the Chongming Island, about 35km away from the sub-system. The locations of the measuring point (Houjiazhen Meteorological Station) and the selected stretch are illustrated in Figure 7.1.

By definition, the 10-minute maximum sustained wind speed is the highest value among all the averaged wind speed over a 10-minute time span during a typhoon event. To be precise, it indicates a
sustained wind intensity instead of an instantaneous value. It is reasonable to select it as the characteristic wind speed. Admittedly, during a typhoon event, instantaneous extreme wind speed could occur, and it could even reach twice as large as the 10-minute sustained wind speed. However, the instantaneous high wind and consequent instantaneous high wave could merely cause water to flow over the flood defence for seconds. Usually, such a short time span is not enough to cause devastating consequences. In contrast, 10-minute average wind speed could reflect the threats of fierce wind and high wave to the flood defences.

However, this dataset has several defects that might affect the accuracy of the results. For instance, the information of wind direction was not contained in the dataset. Besides, a threshold of the wind speed was already set to filter out the wind that cannot be identified as typhoon.

Moreover, a distance of $35 \text{ km}$ between the measuring point and the study stretch could induce a difference of wind speed. In an advanced version of this study method, more accurate data have to be collected from meteorological stations closer to the target field. Another option is to study on the conversion factor between the two locations. The conversion factor depends on the typhoon track and more importantly, friction by ground. Nevertheless, according to Figure 2.10 and 2.11, it could be observed that the wind speed difference between the two locations has a magnitude of about $1 \text{ m/s}$, 5% - 7% of the extreme wind speed ($15 \text{ m/s} - 20 \text{ m/s}$). The error is acceptable in this study.
7.1. Extreme Wind Speed

7.1.2. POT Approach

The goal was to obtain the extreme wind speed versus return years in the typhoon season. Usually, there exist two approaches to practical extreme value analysis. One approach relies on processing block maxima series. For instance, in many cases, Annual Maxima Series (AMS) are generated, and this method is widely applied. The other method is Peak-Over-Threshold (POT). By setting an appropriate threshold, the data series can be filtered. Those above the threshold are taken into account and processed for extreme value analysis.

In this study, POT was selected as the method rather than annual maxima approach. The reasons are stated as follow. Typhoon is unique, different from extreme water level or extreme wave height. In one year, typhoon may strike Shanghai for one to four times, but in some years, typhoon might be absent. Take the example of Shanghai. In 1973, 1976 and 1978, etc., none impacted Shanghai, while in 1990, typhoon struck Shanghai for four times. Therefore, extreme wind speed of typhoon does not happen on a yearly basis. In this case, methods considering maxima in a fixed time span (e.g., one year) conceal the information that typhoon would possibly happen for several times in the time span.

Practically, POT is also a better solution in this case. Detailed dates about wind speed of typhoon was still vague even though 47 sets of characteristic wind speed were collected. It could be a solution to use AMS that the wind speed in the absent years could be assumed in insignificant magnitude such as 5m/s. However, it is not clear how much the results would be affected by the assumptions, but the
effects are bound to exist.

A better solution could be to take 2-year or 3-year maxima instead of annual maxima. Then the goal could be achieved by transforming frequency of occurrence every 2 or 3 year to frequency per year. Nevertheless, by taking 2-year or 3-year maxima in the period of 37 years (1971-2007), only 18 or 12 sets of wind speed could be utilised. It is of no sufficiency to implement a reliable extreme value analysis with them [31].

7.1.3. Computational Model: Bayes

To perform an extreme value analysis, a computational model, namely Bayes, can be adopted. The software is able to perform both POT and AMS analysis. Since the POT is chosen for this case, the theories and principles behind Bayes to process POT analysis is focussed on in this section.

Computational Steps

In order to conduct a POT process, following steps have to be taken [32]:

- **Step I**
  Prescribe a threshold of wind speed over which a set of wind is classified as a typhoon event. Usually, the choice of the threshold should be rather cautious, for various thresholds can result in distinct distributions of the extreme value. If the threshold was set too low, the results would not be informative enough to indicate the distribution of the extreme values. However, if it was set too high, insufficient data would lead to great uncertainty.

- **Step II**
  Use the threshold to filter harmless wind speed, and select a representative wind speed (e.g., 10-minute maximum sustained wind speed) for each typhoon.

- **Step III**
  Sort the typhoon events by ranking their representative wind speed that exceeds the threshold from 47 to 1 where the No. 47 is the typhoon with highest wind speed.

- **Step IV**
  Determine the exceedance probability by applying the following empirical formula [33]:

\[
1 - \frac{i}{N + 1} \cdot \left( \frac{N}{D} \right)
\]

(7.1)

in which
- \(N\) = total number of simulations (= 47 typhoon events)
- \(i\) = ranking of the typhoon
- \(D\) = number of years (= 37 years)
- \(F\) = fetch (\(m\))

- **Step V**
  Derive a marginal distribution function to extrapolate the results to more extreme situations.

In accordance with the steps, the main computational process of Bayes are twofold. The first is to rank the data exceeding the threshold (Step I - Step IV), while the other process is to fit it to a suitable parametric distribution function (Step V).
Input
The input for Bayes is a file containing peak values in a time span. In this case, peak values are the 47 sets of 10-minute maximum sustained wind speed. Another input should be the number of years in which recorded typhoons happened.

Moreover, a threshold has to be prescribed by users. In this dataset, the input wind speed are already high enough to be counted as typhoon. It means the 47 sets of wind speed were already peak value higher than an implicit threshold. Hence, one thing was certain that it was reasonable to set a "fake" threshold which is smaller than the smallest wind speed. This "redundant" step is to guarantee complete inputs for Bayes’s executing. However, in the light of the essence of the POT, it would be also plausible to prescribe higher threshold. In this way, the results would better reflect the distribution of the extreme class, but in a risk of less certainty of the results, because fewer data beyond the threshold could be invoked for calculation.

Output
The output of Bayes is simply probability distributions which are able to indicate the exceedance probability of the inputs in the extreme intervals. Specifically, optimal parameters of distribution are determined and shown. Moreover, it also returns the 5% and 95% confidence intervals of the results.

Parametric Distribution Function
In this module, there are four parametric probability distribution functions as typical candidates for POT analysis, i.e., exponential distribution, Generalised Pareto Distribution (GPD), Left-Truncated Weibull Distribution (LTWD), and Pareto distribution. Essences behind them are shown in Appendix C.1.

Parameter Estimation
In order to estimate the parameters of the four probability distributions that best fit the inputs. Bayes allows two estimators. A classical method is maximum-likelihood estimator (MLE), while as the name of software implies, Bayesian analysis is also embedded. The two estimators are introduced in Appendix C.2.

7.1.4. Results and Discussions
In this section, results of the simulation for the typhoon wind speed at the target stretch are given and discussed.

Probability Distribution
By applying POT method, exceedance probability distribution of extreme wind speed is illustrated in Figure 7.2. In the figure, exponential distribution, left-truncated Webibull distribution, and Pareto distribution are present to fit the observations. Generalised Pareto distribution is absent, because Bayes program does not allow the shape parameter of GPD to be less than -0.5 or greater than 0.5.

In the light of the figure, LTWD apparently best fits the dataset. However, if time permits, sophisticated procedure should be taken to prove this conclusion. An optional idea could be to apply the least squares method: the type of probability distribution with smallest difference between observations and them could be called "best fit". However, they were beyond the scope of this study.
Extreme Wind Speed

The table 7.2 presents the extreme wind speed of different return years simulated by applying maximum likelihood estimator and Bayesian estimator. As is shown in the table, MLE returns the lowest wind speed estimation, and Bayes with MCMC returns the highest. In this study, it is not necessary to discuss which method could offer a better estimation, as the difference of the results, 1m/s - 3m/s is not considerable in this loop.

Table 7.2: Extreme wind speed estimation by different estimation methods (m/s)

<table>
<thead>
<tr>
<th>Method</th>
<th>Return Year</th>
<th>10</th>
<th>50</th>
<th>100</th>
<th>200</th>
<th>500</th>
<th>1,000</th>
<th>5,000</th>
<th>10,000</th>
</tr>
</thead>
<tbody>
<tr>
<td>MLE</td>
<td>5th percentile</td>
<td>15.2</td>
<td>16.6</td>
<td>17.1</td>
<td>17.4</td>
<td>18.0</td>
<td>18.3</td>
<td>18.9</td>
<td>19.2</td>
</tr>
<tr>
<td></td>
<td>50th percentile</td>
<td>15.9</td>
<td>17.5</td>
<td>18.1</td>
<td>18.6</td>
<td>19.2</td>
<td>19.5</td>
<td>20.4</td>
<td>20.7</td>
</tr>
<tr>
<td></td>
<td>95th percentile</td>
<td>16.9</td>
<td>19.1</td>
<td>19.8</td>
<td>20.5</td>
<td>21.3</td>
<td>21.9</td>
<td>23.1</td>
<td>23.5</td>
</tr>
<tr>
<td>Bayes’</td>
<td></td>
<td>16.0</td>
<td>17.8</td>
<td>18.4</td>
<td>19.0</td>
<td>19.8</td>
<td>20.3</td>
<td>21.6</td>
<td>22.1</td>
</tr>
<tr>
<td>Bayes’ with MCMC*</td>
<td></td>
<td>16.1</td>
<td>18.0</td>
<td>18.7</td>
<td>19.4</td>
<td>20.4</td>
<td>21.1</td>
<td>22.9</td>
<td>23.7</td>
</tr>
</tbody>
</table>

* MCMC stands for Markov Chain Monte Carlo simulation, a numerical method to assist Bayesian estimation.

The results by Maximum Likelihood Estimator (MLE) is visualised in the figure 7.3, and the Bayes’ Module returns the cumulative probability distribution of it:

\[
F(x) = 1 - \exp[-3.32 \cdot 10^{-6}(x^{4.904} - 1.6 \cdot 10^4)]
\]

(7.2)

Compared with the wind speed taken in the first loop, 100-year wind speed of 33m/s, the estimation in this chapter is much lower, 18.1m/s - 18.7m/s. By observing the 5th and 95th percentile, a standard deviation is about 0.5m/s for the 100-year condition. It is also much smaller than the assumption of 5m/s. Besides, the distribution of extreme wind speed in each condition is no long symmetrical normal distribution. Instead, it is inclined to the lower value with a long tail at high value, so the lognormal distribution or the Rayleigh distribution could be better assumptions, which can be further studied on in the future.
Furthermore, Dr. Wu, Z. and his colleagues suggested that the extreme wind speed in Shanghai follows Type III (Weibull) distribution [30]. Meanwhile, they also concluded the following formula:

\[ F(x) = \exp \left[ -\left( \frac{x - 23.1902}{11.8174} \right)^{4.8878} \right] \]  

(7.3)

in which, \( x \) represents wind speed. Through this formula, extreme wind speed corresponding to return periods can be estimated and are shown in Figure 7.4. By comparison, the two sets of results are consistent with each other. In general, the clusters of wind speed concluded in this study slightly (about
±7%) differ from the compared set. The ±7% does not affect the results significantly. Specifically, the differences of the overflow/overtopping discharge caused by the 7% is explained in Appendix A.1. Hence, it was concluded that the derived probabilistic distribution of extreme wind speed is adoptable. Finally, owing to the distance between the target stretch and the measuring point, the extreme wind speed should be multiplied by a reduction factor:

\[ U_i = m_\lambda \cdot U_{i,\lambda} \]  

(7.4)

in which:
- \( m_\lambda \) = reduction factor
- \( U_i \) = wind speed corresponding to return period of \( i \) years at the target stretch
- \( U_{i,\lambda} \) = wind speed corresponding to return period of \( i \) years in a distance of \( \lambda \)

In this study, the reduction factor was not studied on. Reasonably, it is related to the distance, friction of the ground, and typhoon track. Here, according to Figure 2.11, a plausible estimation of \( m \) could be 0.9-0.95. Further study about it is recommended.

**Effect of Threshold**

In the preceding sections, it has been emphasised that threshold might be a factor to affect the results. Therefore, a sensitivity analysis was performed by taking different threshold. The results can be seen in Table 7.3.

**Table 7.3: Relation between extreme wind speed and threshold with maximum-likelihood estimator**

<table>
<thead>
<tr>
<th>Threshold ((m/s))</th>
<th>Return Year</th>
<th>Number of observations above threshold</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10</td>
<td>50</td>
</tr>
<tr>
<td>7.2</td>
<td>15.9</td>
<td>17.5</td>
</tr>
<tr>
<td>8</td>
<td>15.9</td>
<td>17.6</td>
</tr>
<tr>
<td>9</td>
<td>16.0</td>
<td>17.7</td>
</tr>
<tr>
<td>10</td>
<td>16.0</td>
<td>17.6</td>
</tr>
<tr>
<td>11</td>
<td>16.0</td>
<td>17.8</td>
</tr>
<tr>
<td>12</td>
<td>16.0</td>
<td>17.9</td>
</tr>
<tr>
<td>13</td>
<td>16.1</td>
<td>17.9</td>
</tr>
<tr>
<td>14</td>
<td>16.2</td>
<td>17.9</td>
</tr>
<tr>
<td>15</td>
<td>16.4</td>
<td>17.8</td>
</tr>
<tr>
<td>16</td>
<td>16.5</td>
<td>17.8</td>
</tr>
</tbody>
</table>

By adjusting threshold from 7.3\(m/s)\) to 13\(m/s)\), the estimation of extreme wind speed only slightly fluctuates by less than ±5%. As is explained, 5% difference is still allowable in this study. However, from 13\(m/s)\) onwards, the estimated wind speed drop significantly with the rise of threshold.

The results imply that the estimation at the high-wind-speed range (>13\(m/s)\)) is different from that at the low-wind-speed range (<13\(m/s)\)). This conclusion is consistent with the observations from the figure 7.5a. Within the range between 7\(m/s\) - 13\(m/s\) the LTWD fits the data points well, but the six data points with wind speed higher than 15\(m/s\) deviate away from the red curve.

Take a close look at the six data points in Figure 7.5b. The red curve has a bending trend towards the lower speed, which appears to approach an upper limit. However, it is not common to observe a speed limit in the typhoon condition. A research revealed that the 100-year typhoon wind speed at East China
Sea can be up to 45 m/s [34], which is much higher than the estimation (18.1 m/s). A possible reason for the limit wind speed might be the reduction effects by high-rise buildings in the urban area. In Shanghai, a leading portion of buildings are taller than 10 m, altitude of measuring wind speed. When typhoon approached the city, the tall barriers force the wind to bypass and even to eddy. So, the measurements as well as the distribution in this section might be biased. The actual situation at Houjiazhen is complex, and the on-site wind speed at the selected stretch might be even more complicated, as it is surrounded by more high-rise buildings at the downtown.

Update of the wave Height

With updated extreme wind speed and the assumption that the extreme wind speed and the extreme water level are fully correlated, the corresponding wave heights are given in Table 7.4. In comparison with the first assumption of 0.7 m, the wave height was reduced significantly to about 0.3 m - 0.5 m.

Table 7.4: Updated wave height estimation (m)

<table>
<thead>
<tr>
<th>Method</th>
<th>Return year</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10</td>
</tr>
<tr>
<td>MLE</td>
<td>0.32</td>
</tr>
<tr>
<td>Bayes’</td>
<td>0.32</td>
</tr>
<tr>
<td>Bayes’ with MCMC</td>
<td>0.32</td>
</tr>
</tbody>
</table>

7.2. Wind Direction

In the first loop, wind direction was simply assumed to be perpendicular to the orientation of the flood defences, and fetch was also simply assumed to be a conservative value. Actually, both factors have large influences on the overtopping probability. The importance of them have been proved in Section 6.3.
7.2.1. Contributions of Wind Direction

Direction-induced Conditional Failure Probability

Figure 7.6 is a flow chart in which the wind direction contributes to the results by adding conditions to the reliability analysis. The wind does not necessarily come from one direction, contrary to the assumption in the first loop. As is shown in the figure, the overtopping/overflow probability of a certain segment is calculated for each wind direction respectively, after which the failure probabilities for each direction are combined into one for the segment. This additional procedure, in practice, lowers the failure probability to a large extent.

\[
P_f = \sum_{i=1}^{16} P(f \mid \psi_i)P(\psi_i)
\]  

(7.5)

in which:

- \(P(\psi_i)\) = probability of occurrence of wind direction \(\psi_i\) (e.g., N, NNW, NE)
- \(P_f\) = failure probability of one failure mode for one segment
- \(P(f \mid \psi_i)\) = failure probability given wind condition \(\psi_i\)

Contribution to Fetch

Another effect of considering the wind direction lies in the fetch. Instead of the assumption of a single wind direction which is perpendicular to the orientation of the flood defences, wind from various directions are able to affect the distance of wave development. Taking these wind directions into account, the conservative assumptions of fetch length could be replaced with effective fetch.

Contribution to Wave Loading

Waves are developed by wind in different directions. The waves with different incident angles have different loading effects on the flood defences.
7.2. Wind Direction

7.2.2. Wind Direction in Shanghai

According to the research of Zhao, L. et al. [35], occurrence probability of wind direction is summarised in Figure 7.7. The site of the measurements is close to Wusongkou, estuary of the Huangpu River, 10 km away from the selected stretch. Admittedly, differences of wind direction due to the 10 km is inevitable, and also, the general wind direction distribution of the extreme wind might not best represent the wind direction of a certain typhoon event. However, it was a sound approximation to start with, and the research can be furthered given better data and perception.

According to the figure, the dominating extreme wind direction is the east and the north with 0.1576 and 0.1414 probability of occurrence respectively. In contrast, there is little occurrence probability of the extreme wind coming from the range between WSW and SSE.

<table>
<thead>
<tr>
<th>Wind direction</th>
<th>Occurrence probability</th>
</tr>
</thead>
<tbody>
<tr>
<td>N (0°)</td>
<td>0.1414</td>
</tr>
<tr>
<td>NNE</td>
<td>0.1370</td>
</tr>
<tr>
<td>NE</td>
<td>0.0894</td>
</tr>
<tr>
<td>ENE</td>
<td>0.1082</td>
</tr>
<tr>
<td>E (90°)</td>
<td>0.1576</td>
</tr>
<tr>
<td>ESE</td>
<td>0.0682</td>
</tr>
<tr>
<td>SE</td>
<td>0.0611</td>
</tr>
<tr>
<td>SSE</td>
<td>0.0000</td>
</tr>
<tr>
<td>S (180°)</td>
<td>0.0000</td>
</tr>
<tr>
<td>SSW</td>
<td>0.0000</td>
</tr>
<tr>
<td>SW</td>
<td>0.0000</td>
</tr>
<tr>
<td>WSW</td>
<td>0.0000</td>
</tr>
<tr>
<td>W (270°)</td>
<td>0.0000</td>
</tr>
<tr>
<td>WNW</td>
<td>0.0430</td>
</tr>
<tr>
<td>NW</td>
<td>0.0810</td>
</tr>
<tr>
<td>NNW</td>
<td>0.1130</td>
</tr>
<tr>
<td>OMNI</td>
<td>1.0000</td>
</tr>
</tbody>
</table>

Figure 7.7: Wind-rose diagram of extreme wind at the estuary of the Huangpu River (Zhao, L. et al., 2005)

According to the figure, the dominating extreme wind direction is the east and the north with 0.1576 and 0.1414 probability of occurrence respectively. In contrast, there is little occurrence probability of the extreme wind coming from the range between WSW and SSE.

7.2.3. Effective Fetch

In the same way, 16 wind directions were defined with an angle interval of 22.5°, namely, 0°, 22.5°, 45°, ..., 337.5°, etc., which respectively represent N, NNE, NE, ..., NNW, etc. The wind directions actually represent the midpoints of a width of 22.5°. For example, the direction 22.5° stands for the sector 11.25° - 33.75°.

Corresponding to the 16 wind directions, fetch length differ. Here, effective fetch was adopted, because waves are generated and developed by not only wind of one direction but also that of adjacent directions. The contribution of adjacent directions are distinct, so the effective fetch entails a weighing parameter by using the following formula [36]:

\[
F = \frac{\sum_{i=1}^{15} R(\phi_i) \cos^2(\phi_i)}{\sum_{i=1}^{15} \cos(\phi_i)} \tag{7.6}
\]
Table 7.5: Wind and fetch data for the floodwall section 4

<table>
<thead>
<tr>
<th>Wind direction</th>
<th>Water depth (m)</th>
<th>Effective fetch (m)</th>
<th>Wind speed (m/s)</th>
<th>Occurrence probability</th>
</tr>
</thead>
<tbody>
<tr>
<td>N (0°)</td>
<td>1035</td>
<td>1035</td>
<td>0.1414</td>
<td></td>
</tr>
<tr>
<td>NNE</td>
<td>772</td>
<td>772</td>
<td>0.1370</td>
<td></td>
</tr>
<tr>
<td>NE</td>
<td>209</td>
<td>209</td>
<td>0.0894</td>
<td></td>
</tr>
<tr>
<td>ENE</td>
<td>18</td>
<td>18</td>
<td>0.1082</td>
<td></td>
</tr>
<tr>
<td>E (90°)</td>
<td>0</td>
<td>0</td>
<td>0.1576</td>
<td></td>
</tr>
<tr>
<td>ESE</td>
<td>0</td>
<td>0</td>
<td>0.0682</td>
<td></td>
</tr>
<tr>
<td>SE</td>
<td>0</td>
<td>0</td>
<td>0.0611</td>
<td></td>
</tr>
<tr>
<td>SSE</td>
<td>0</td>
<td>0</td>
<td>0.0000</td>
<td></td>
</tr>
<tr>
<td>S (180°)</td>
<td>9.0</td>
<td>9.0</td>
<td>0.0000</td>
<td></td>
</tr>
<tr>
<td>SSW</td>
<td>0</td>
<td>0</td>
<td>0.0000</td>
<td></td>
</tr>
<tr>
<td>SW</td>
<td>0</td>
<td>0</td>
<td>0.0000</td>
<td></td>
</tr>
<tr>
<td>WSW</td>
<td>0</td>
<td>0</td>
<td>0.0000</td>
<td></td>
</tr>
<tr>
<td>W (270°)</td>
<td>0</td>
<td>0</td>
<td>0.0000</td>
<td></td>
</tr>
<tr>
<td>WNW</td>
<td>555</td>
<td>555</td>
<td>0.0430</td>
<td></td>
</tr>
<tr>
<td>NW</td>
<td>894</td>
<td>894</td>
<td>0.0810</td>
<td></td>
</tr>
<tr>
<td>NWW</td>
<td>1153</td>
<td>1153</td>
<td>0.1130</td>
<td></td>
</tr>
</tbody>
</table>

An example corresponding to wind direction N for the floodwall section 4 is demonstrated in Figure 7.8. Originating from the point A, a bundle of lines are considered, centering around the wind direction. The included angle between one of the lines and the central line is defined to be $\phi_i$. With a step size of 6°, the included angles range from -42° to 42°. Therefore, there are in total 15 lines in the bundle, for each
of which, a fetch \( R(\phi_i) \) is specified.

The formula 7.6 can be applied to all the 16 wind directions and to all the flood defence segments. An example of the wind and fetch data for the floodwall 4 is listed in Table 7.5.

Besides the effective fetch and wind direction that are explained in this section, wind speed and water depth should also be specified particularly for each wind direction. The wind speed differs in different directions. Besides, the average water depth differs in various wave propagation directions owing to the complex bottom topography of the river. However, because of the data limit, the water depth and wind speed were provisionally assumed to be isotropic. Further work is recommended in the future to substitute the assumptions with anisotropic data.

### 7.2.4. Reduction Factor

In order to involve the effect of wind directions on wave loading, the limit state function of overtopping/overflow should be modified by adding a reduction factor \( \gamma_\beta \). Corresponding to each set of wind direction and effective fetch, a \( \gamma_\beta \) can be specified in the following formula for overtopping/overflow [24]:

\[
Z = q_c - q
\]  

When \( z_0 \leq z_c - \Delta l \),

\[
q = m_{os} \cdot \sqrt{g(H)^3} \cdot \exp \left[ -3.0 \frac{(z_c - \Delta l - z_0)}{H} \frac{1}{\gamma_\beta} \right] \times 1000
\]  

\( q = 0 \), given \( \beta > 90^\circ \)

When \( z_0 > z_c - \Delta l \),

\[
q = \left[ m_{ot} \cdot 0.55 \sqrt{-g(z_c - \Delta l - z_0)^3} + m_{os} \cdot \sqrt{g(H)^3} \right] \times 1000
\]  

\( \sqrt{g(H)^3} = 0 \), given \( \beta > 90^\circ \)

in which

- \( q_c \) = critical discharge \((l/m/s)\)
- \( q \) = average overflow/overtopping discharge \((l/m/s)\)
- \( H \) = significant wave height, calculated by the Formula 2.2 \((m)\)
- \( z_c \) = crest level \((m)\)
- \( z_0 \) = local water level \((m)\)
- \( \Delta l \) = land subsidence \((m)\)
- \( g \) = gravitational acceleration \((9.81 m/s^2)\)
- \( m_{os} \) = model factor overtopping \( \sim logN(0.34, 0.09^2) \)
- \( m_{ot} \) = model factor flow \( \sim N(1.1, 0.3) \)

In addition,

- \( \gamma_\beta \) = reduction factor due to the obliqueness of waves
- \( \beta \) = angle between the propagation direction of waves and the axis perpendicular to structures

Specifically, the factor can be approximated as follows [24]:

\[
\gamma_\beta = \begin{cases} 
1.0, & \text{if } \beta \leq 20^\circ \\
\cos(\beta - 20^\circ), & \text{if } 20^\circ < \beta \leq 90^\circ, \text{with a minimum of } \gamma_\beta = 0.7
\end{cases}
\]  

(7.10)
7.3. Wind Duration

Duration of typhoon and extreme wind speed is able to affect the reliability of the flood defences system. It is obvious that a storm or typhoon prevailing for longer period could cause more amounts of water flowing over the crests. It is also obvious that a gust would not destroy the flood defences even if it prevails with an extremely high wind speed.

However, this parameter does not play a role in the limit state functions of overtopping/overflow. In this section, the effect of the typhoon duration is briefly discussed.

7.3.1. Criteria of Overflow/overtopping Failure

At the end of the first loop, overflow/overtopping was proved to be an essential hazard. However, the failure was merely defined by a constant critical discharge ($10 \frac{l}{m/s}$). According to the PC-Ring manual [24], the failure of the flood defences should be defined as either of the following two scenarios (seen in Figure 7.9):

- Water flows over crests and into backlands, driven by waves or high water level that exceeds the crest. The amount of it is sufficient to lead to devastating inundation depth.
- Waves and water impulse structures rearward and cause breaching, so that the structures are not able to maintain their flood-retaining functions.

![Fault tree of overtopping/overflow](Vrouwenvelder,A., et al., 1999)

The consequence of the two scenarios are also different. Since consequence analysis was excluded from this thesis, it is of no necessity to be discussed in detail. As an overview, a key difference of the scenarios is illustrated in Figure 7.10. The red dots represent the threshold of failure. The shadow area stands for the amount of water flowing into the backland of the flood defences.

In the first scenario of water inundation, the effect is merely restricted within the period when overtopping/overflow is proceeding ($q > q_c$). In comparison, in the latter scenario, water keeps flowing through the breaches since the failure occurs. Hence, the second type of failure is more destructive than the first. Besides, the first scenario is more sensitive to the wind duration: the longer it prevails, the more damage it could result in.

Additionally, the latter scenarios is also influenced by emergency measures. For example, the structures can be repaired after breaches are detected, and importantly, the duration of it can affect the
7.3. Wind Duration

Figure 7.10: Scenarios of overflow/overtopping failure. The overtopping/overflow discharge $q$ on the y-axis represents the strength term in Formula 3.1. The red dashed lines stand for the threshold of failure above which $q > q_c$ (i.e., $Z < 0$).

process of inundation. A quick reparation against breaching could effectively relieve the flooding consequences.

The floodwalls and floodgates along the selected stretch are mostly concrete or steel structures, and the ground surface behind them are also well protected due to the usage of the backlands (e.g., schools, warehouses, communities, etc.). Therefore, the likelihood of the second scenarios is limited.

7.3.2. Typhoon Duration and Inundation Depth

Considering the former scenario, how much damage a typhoon-induced event of overtopping/overflow could cause? The result is highly associated with the terrains behind the defences and the average discharge.

Take the floodwall section 4 for example. Behind the floodwalls is a harbour storage belonging to a general cargo terminal. According to the type of freighters berthing at the terminal, which is observed on the Google map, the main type of cargo is building materials. Although the topographic data was not available, assumptions could be made and is shown in Figure 7.11. After overtopping or overflow, the flood inundates the storage yard and the neighborhood afterwards.

An approximation of inundation depth was performed by using a very simply formula:

$$d_{in} = \frac{Q t}{A_e} \cdot \frac{1}{1000} \quad (7.11)$$

For each unit width,

$$d_{in} = \frac{q t}{L_e} \cdot \frac{1}{1000} \quad (7.12)$$

in which

$Q$ = average overflow/overtopping discharge ($l/s$)
$q$ = average overflow/overtopping discharge per unit width ($l/m/s$)
$d_{in}$ = inundation depth ($m$)
\[ t = \text{wind duration (s)} \]
\[ A_e = \text{inundated area behind the floodwall (m)} \]
\[ L_e = \text{length of inundated area behind the floodwall (m)} \]

The wind speed taken in this study was 10-minute maximum sustained wind speed, which represents the maximal 10-minute average wind speeds. So, the first assumption of wind duration could be 10 minutes. The effective width was measured to be 500m beyond which is the neighborhoods. Most of the ports should be surrounded by separating walls through which water could hardly flow. Besides, the actual overflow/overtopping discharge at the floodwall section 4 is about 400l/m/s in the 1,000-year water level condition. Finally, it returns an inundation depth of 0.5m, which is a rather large depth.

Wind duration is usually larger than 10 minutes, because the slightly lower wind speed than the maximum could also lead to inundation, although the induced wave height is correspondingly lower. Figure
7.12 shows the inundation depth in the condition of various freeboard and wind duration. Notice that wave duration was assumed to be equal to wind duration.

According to the figure, in the 1,000-year water level condition and 3×10 minutes, the inundation depth can be over waist-deep 1.2m.

Figure 7.13 shows the relative damage rate\(^1\) of properties. Guangzhou and Shenzhen are two big cities with similar status as Shanghai. Hence, they can be references to estimate the damage level of properties owing to flooding. The damage rate is approximately 20% for inventories in the condition of 0.5m - 1.0m inundation depth, which is quite high damage to the region.

7.3.3. Typhoon Duration and Critical Discharge

In return, the critical discharge \( q_c \) can be re-estimated by rewriting the equation 7.12:

\[
q_c = 1000 \cdot \frac{L_c d_{in,E}}{t}
\]  

(7.13)

The \( d_{in} \) is replaced by an allowable inundation depth, which reflects the allowable consequence in the area. Therefore, the usage, the topography, and the population of the backland together with typhoon duration and affected area should be taken into consideration to specify the critical discharge.

However, it is worth mentioning that this equation only involves the second scenario under the assumption that breaching could hardly occur at the structures.

7.4. Discussions

7.4.1. Updated Overtopping/overflow Probability

The updated overtopping/overflow probability was concluded and compared with that from the 1st loop. The results are shown in Table 7.6. It can be seen that the failure probability dropped significantly, contributed to by less conservative wind speed, fetch length, and the consideration of wind direction.

In any single segment, the yearly overtopping/overflow probability drops by a factor of 10\(^3\). The section 1, section 2, and the gate a still contribute the most to the system because of their low crest levels, surpassed by the water level.

\(^1\)The fraction of the amount of damage (i.e., repair cost) to the maximum economic damage of the segments.
Table 7.6: Updated yearly overtopping/overflow probability compared with that from the 1st loop

<table>
<thead>
<tr>
<th>Segment</th>
<th>1st Loop</th>
<th>2nd Loop</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section 1</td>
<td>0.31120</td>
<td>0.00352</td>
</tr>
<tr>
<td>Section 2</td>
<td>0.30365</td>
<td>0.00338</td>
</tr>
<tr>
<td>Section 3</td>
<td>0.14580</td>
<td>0.00132</td>
</tr>
<tr>
<td>Section 4</td>
<td>0.05693</td>
<td>0.00054</td>
</tr>
<tr>
<td>Section 5</td>
<td>0.07420</td>
<td>0.00063</td>
</tr>
<tr>
<td>Section 6</td>
<td>0.06745</td>
<td>0.00051</td>
</tr>
<tr>
<td>Section 7</td>
<td>0.05051</td>
<td>0.00037</td>
</tr>
<tr>
<td>Section 8</td>
<td>0.05095</td>
<td>0.00041</td>
</tr>
<tr>
<td>Gate a</td>
<td>0.29419</td>
<td>0.00329</td>
</tr>
<tr>
<td>Gate b</td>
<td>0.14709</td>
<td>0.00130</td>
</tr>
<tr>
<td>Gate c</td>
<td>0.04453</td>
<td>0.00038</td>
</tr>
<tr>
<td>Gate d</td>
<td>0.04888</td>
<td>0.00044</td>
</tr>
<tr>
<td>Gate e</td>
<td>0.04903</td>
<td>0.00042</td>
</tr>
<tr>
<td>Combined</td>
<td>0.31120</td>
<td>0.0035  - 0.0165</td>
</tr>
<tr>
<td>System $P_f$</td>
<td>0.31 - 0.36</td>
<td>0.033 - 0.055</td>
</tr>
</tbody>
</table>

The fragility curve of the floodgate a was updated, and it is compared with the fragility curve in the first loop, which is shown in Figure 7.14. The curve was narrowed, because with lower wave height, the impacts of water level became larger on the results. The curve was also shifted rightwards. Failure starts to occur at the 50-year water level of around 5.6m and reaches nearly 1.0 at the 500-year water level of around 6.3m. This stretch should have been designed for the 1000-year water level condition according to the design standard [15]. However, the conditional probability of 1.0 at the water level condition of 500 years showed a contradiction. Considering more segments and more failure modes, the system might even fail at less than 100-year water level. Therefore, the failure probability was overestimated, or the flood defences had met the safety standard prescribed in the design code.

Figure 7.14: Comparison of fragility curves of overtopping/overflow in Gate a
By taking elementary upper bound $\Sigma R$ and the lower bound $\max(R)$, the failure probability of the sub-system decreased massively from 0.31-0.36 per year to 0.033-0.055 per year by 95%. It reaches once 20-30 years. The flood defence along the Huangpu River was last reinforced and heightened extensively during 1998 and 2005 [37]. Hence, the historical records after the reinforcement are not sufficient to prove whether the failure probability is large or not. However, as a proof of concept, the once 20-30 years can be compared with the 1000-year standard of water level, and the failure probability still seems higher than expected.

Besides, a comparison with the previous estimations is shown in Table 7.7. The overtopping/overflow probability was reduced to a bit less than the closure failure of floodgates. Regardless of the vague piping mechanism due to limited information, the closure failure has become the most influential failure modes. It implies a priority to study further on it.

Table 7.7: Updated comparison of failure mechanisms

<table>
<thead>
<tr>
<th></th>
<th>Overtopping/Piping</th>
<th>Piping</th>
<th>Structural Failure</th>
<th>Closure failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Combined $R_f$</td>
<td>3.5E-03 - 1.7E-02</td>
<td>1.9E-02</td>
<td>7.0E-5</td>
<td>3.3E-2</td>
</tr>
<tr>
<td>Ranking</td>
<td>3rd</td>
<td>2nd</td>
<td>4th</td>
<td>1st</td>
</tr>
</tbody>
</table>

### 7.4.2. Influence of New Assumptions

In summary, conservative assumptions were made on the wind properties in the first loop:

- Wind speed $U$ was assumed to be homogeneously high in various extreme conditions, following $N(33\,m/s, 5m/s)$.

- Wind direction and incident waves were always perpendicular to the orientation of any of the flood defence segments. The reduction factor caused by incident angles was not incorporated in the limit state function.

- Fetch was most conservatively assumed to be 2,000$m$ for every segment.

Based on the progress in this chapter, the updated assumptions were less conservative:

- Corresponding to each return period, the wind speed $U$ is distinct, which is shown in Table 7.2. Its extreme condition accords with that of the water level. For instance, 100-year wind speed and 100-year water level are coupled, and 1,000-year wind speed and 1,000-year water level are coupled, etc.

- 16 wind directions were defined, each of which owns its own occurrence probability. Besides, the reduction factor $\gamma_w$ was specified in the limit state function.

- The fetch length varies in different directions. Adjacent directions also contribute to the length of fetch, which was specified into "effective fetch".

The table 7.8 indicates the influence of wind properties by adding them once a time. By comparing the No.2, No.3, and No.4 with the first set, each of the three brings significant drop to the failure probability by overtopping/overflow. Among them, the influence of wind speed is the largest, as the decrease from No.1 to No.2 is the greatest. The influence of the fetch comes the second, and that of wind directions is the smallest among the three. This finding is consistent with the conclusions shown by the influence factor $a$ in Section 6.3.
In the set No.4, the effects of effective fetch on the three sections are different from one another. It’s because the locations and corresponding geometrical characteristics of the three sections lead to various effective fetches.

Except for in No.4, there is no visible difference of influence among different sections. Section 1 was identified to be an overflow-dominated section, while overtopping prevailed along the other two sections. Hence, the result implies no difference of influences between overtopping and overflow. This is abnormal, because the wind properties related with wave generation should have had more influence on overtopping than on overflow. However, it can be explained by tracing to the limit state function of the overflow, the function 3.3. Given $z_0 > z_c - \Delta l$, overflow was triggered:

$$q = 1000 \cdot m_{at} \cdot 0.55\sqrt{-g(z_c - \Delta l - z_0)^3} + 1000 \cdot m_{at} \cdot \sqrt{g(H)^3}$$

The latter term contains the effects of the wave height. According to the table, the role of this term is so important that the wave becomes the predominant factor in determining the discharge. In the light of this, updating wind properties played as important role in Section 1 as in Section 4 and 8. This is consistent with the findings in the section 6.3.1: The discharge caused by the wave-affected term is 8 times higher than that caused by the excessive water level.

### 7.4.3. Discussions about Wind Speed

**Lower Wind Speed in Shanghai than in the Netherlands**

The distribution of the extreme wind speed in Shanghai has been estimated. A comparison with the wind speed of the storms in the DeBilt is presented in Figure 7.15. The data are quoted from *Extreme wind statistics for the Hydraulic Boundary Conditions for the Dutch primary water defences* [38]. Wind speed in DeBilt was chosen because the distance of it from the coast is similar to that of the sub-system in Shanghai. The distance is about 40km-50km.

As is shown in the graph, wind speed in DeBilt is much higher than that in Shanghai, which seems contradictory to the expectation. However, it can be explained in two aspects.

The genesis of typhoon requires sufficiently warm sea surface temperatures and high humidity [39]. That is why typhoons were usually formed over the ocean in summer. Different from the storms in the Netherlands, typhoon center travels. Once it reaches the land, the decreased temperature and humidity leads to a rapid decay of typhoon. As a result, the wind speed declined significantly after typhoon’s landing, and the measured wind speed onshore is usually higher than that offshore.

The other reason is that the roughness of the city decreases the wind speed. Especially in Shanghai, a great quantity of buildings and other infrastructures lead to large friction against wind.
7.4. Discussions

Figure 7.15: Comparison of extreme wind speed between the selected stretch in Shanghai and DeBilt in the Netherlands (data source: Caires, 2009)

Sensitivity Analysis of Wind Speed

However, the reliability of the original data could be also a reason why the estimated wind speed is not in line with the expectation. In order to evaluate how the wind speed can affect the results. A sensitivity analysis was performed by adopting higher wind speed than the estimation. The wind speed distribution in DeBilt and two imaginary distributions were taken for comparison. The distributions of the wind speed are shown in Figure 7.16.

Figure 7.16: Adopted distributions of wind speed
In order to assess the influence of higher wind speed, the wind speed was adjusted, while other variables such as fetch, wind direction, water level, etc. remained the same. The failure probability of Section 1, Section 4, and Section 8 under the condition of assumed higher wind speed are shown in Table 7.9. Failure probability with the assumption of the imaginary sets of wind speed and the set in DeBilt are respectively compared with the red set. The increase in Section 1 is smaller than the other two sections, because Section 1 with lower crest level is affected more by the water level than by wind wave. For the other two floodwall sections, Imaginary A wind speed leads to an increase by a factor of 1.3; the wind speed in DeBilt leads to a rise of a factor of 1.6; Imaginary B wind speed leads to an increase by a factor of 2.0.

**Table 7.9: Influence of higher wind speed.** The red set is with the estimated wind speed in from the Section 7.1 in this loop. The increasing rate are based on this set.

<table>
<thead>
<tr>
<th>Wind Speed Set</th>
<th>Section 1</th>
<th>Section 4</th>
<th>Section 8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shanghai</td>
<td>0.00352 (1.09×)</td>
<td>0.00054 (1.23×)</td>
<td>0.00041 (1.38×)</td>
</tr>
<tr>
<td>Imaginary A</td>
<td>0.00421 (1.20×)</td>
<td>0.00081 (1.50×)</td>
<td>0.00069 (1.68×)</td>
</tr>
<tr>
<td>DeBilt</td>
<td>0.00490 (1.39×)</td>
<td>0.00112 (2.07×)</td>
<td>0.00094 (2.29×)</td>
</tr>
<tr>
<td>Imaginary B</td>
<td>0.308 (87.5×)</td>
<td>0.057 (105.6×)</td>
<td>0.051 (124.4×)</td>
</tr>
</tbody>
</table>

The increase from the 2nd-phase results in red to the the 1st-phase result is by a factor of 100. Moreover, the unconditional failure probability by failure to close floodgates was assumed to be about 0.033, 10 times higher than the red. In comparison, an increase of failure probability by a factor of around 2.0 is not significant. Take a look at Figure 7.16. The wind speed difference is significant among the three curves when the water level condition is above 500 years. However, because of the higher probability of lower wind speed below the 100-year condition, the design point of the wind speed is in that interval. Around the design point, the difference of wind speed among the three curves are small, so the results do not vary significantly by adopting the three curves.

In conclusion, although it was concluded that wind speed is the most influential factor among the wind properties, it is not urgent to validate the data source of wind speed.

### 7.4.4. Wind Properties and Typhoon Characteristics

Unlike the storms sweeping across the Netherlands, typhoon owns unique characteristics, and wind properties might be thus different. In this sub-section, several possibly crucial points are roughly discussed, which could lead to a further reduce of the overtopping/overflow probability given specific studies in the future.

**Typhoons in Shanghai and Storms in the Netherlands**

The Netherlands, located at the northwest corner of Continental Europe, is influenced by strong prevailing westerlies. Especially in winter, the storms prevail in the country, and the main wind direction is the west. This type of wind majorly originates from the differences in heat and air pressure between 30°N and 60°N, and Coriolis effect. As a result, the local wind direction in the Netherlands is usually identical in a certain duration of a storm.

Along the Huangpu River in Shanghai, the type of storms is different from the westerlies in the Netherlands. Typhoons, dependent on the location of formation, are also called hurricanes (near the Amer-
Discussions

Typhoons rotate around their own central axes. Unlike westerlies, the central axes as well as the entire typhoons can travel along a path. They often land in continents, and coastal cities such as Shanghai, Hong Kong, and Tokyo have been often victims. Owing to the movable and rotating features of typhoons, wind speed, wind directions, and their corresponding wind durations vary over the entire typhoon event. The scale and the track of the typhoon can make contributions.

Effects on Wind Properties

In the analysis of extreme wind speed and wind direction, indirect data were adopted. They were measured at meteorological stations and proceeded statistically. For instance, the wind speed taken was the average over 10 minutes; the wind direction was the average of all the historical typhoons. The number itself showed the on-site performance of typhoon and can be utilised despite the essences behind it are implicit.

Considering the complexity of typhoons, wind speed and wind direction can vary tremendously on a minutely basis. Waves can do harm to the flood defences only with a sufficient cumulation of time. Therefore, the scale of the variation of wind properties and the wind duration should be approximated from the perspective of the track and form of typhoons.

Figure 7.17 shows the wind speed distribution from top view. The example was taken from a typhoon near Tokyo. In the center (central axis), the wind speed is relatively low. This zone is defined as “eye” of the typhoon. Usually, the radius of this zone is about 10 km. From the 10-kilometer radius outwards, wind speed becomes more intense and reaches the peak at the radius of around 70 km.

![Spatial distribution of wind speed in a typhoon](Klaver, 2005)
Moreover, due to Coriolis effect, the peak wind speeds are observed at the right (east) of the typhoon in the northern hemisphere [40]. In general, the extreme wind speed that can threats flood defences are distributed over the radius of 20\(k m\)-150\(k m\), right side of the typhoon’s track. The wind direction at a point in a typhoon is a combination of propagation speed and rotational speed, which is illustrated in Figure 7.18. Generally, the wind direction at the peak region is tangent to the circle and parallel to the storm track.

Take the typhoon Fung-wong in September, 2014 for example. The figure 7.19 shows the track of Fung-wong. The white dashed circles represents the bounds for extreme wind speeds where the radius \(R \in (20km, 150km)\). Provided that the bounds were reliable, the stretch was affected by the typhoon for one day. From Sept 23rd 0 a.m. to Sept 23rd 12 a.m., the stretch was covered by extreme wind speed (between small circles and large circles). Furthermore, during the 3 hours from 3 a.m. to 6 a.m., the wind speed approached the peak value, as the stretch is closer to the small circles.

Take a close look at the period when typhoon landed in Shanghai and affected the stretch in Figure 7.20. The wind direction at the stretch varied by up to 120°, because it took a hard turn at around 3:30
a.m. However, between 4:00 a.m. and 4:30 a.m., the typhoon is closest to the stretch, and the wind direction at the stretch is nearly identical during this period.

![Figure 7.20: Track of the typhoon Fung-wong and wind directions at the selected stretch](image)

From 0:00 a.m. to 6:00 a.m., the dynamic characteristics of Fung-wong results in variation of wind speed, wind direction and fetch. Figure 7.21 conceptually shows the variation of them at Section 4 for example. Wind speed and fetch reach the peak at around 4:00 a.m. However, at that time, wind direction is equal to 0, which means the wind blows parallel to the section. In this case, wave’s effects might not be the maximum. Instead, the maximal wave’s effects might appear around 6:00 a.m. when lower wind speed and shorter fetch is combined with the wind direction perpendicular to the section. However, at that time, the Huangpu River was at low tide, so the probability of overtopping/overflow was not high around 6:00 a.m.

In summary, this typhoon affected Shanghai for one day but only threatened the stretch for 3 hours. The wind properties varied significantly during the 3 hours. Besides, the extreme wind speed could last for tens of minutes, during which the wind direction was steady but parallel to the section 4.
Figure 7.21: Variation of wind properties and water level at Section 4 during the typhoon Fung-wong. Wind direction is represented by the angle between the orientation of the structures and wind direction. The negative angle means wind blows away from section 4.

**Closure**

Typhoon Fung-wong is merely one example, but it did offer some information about typhoon that affects Shanghai:

- Typhoon can affect Shanghai for about one day, but the extreme wind speed can last only for tens of minutes.
• Wind speed, wind direction, and corresponding fetch vary significantly with the rotation and propagation of typhoon. The peak values of them might not coincide.

• High waves and high water level might not coincide.

These characteristics of typhoon can be sources of overestimating overtopping/overflow. In this loop, the duration of wind and wind-induced waves was not taken into calculation. However, during a duration of tens of minutes, the extreme wind speed might not cause serious consequence to the backlands unless breaches of the structures occurred. In the duration of about 3 hours, the wind speed does not remain the extreme value. Therefore, the wind speed adopted for the 3 hours should be lowered.

Moreover, typhoon rotates and propagates significantly, leading to drastic variation of wind properties. In the example of Fung-wong, there were phase differences among the wind properties, so peak values of them did not coincide. However, in this study, an assumption was that the estimated extreme wind speed could coincide with any wind directions including the critical directions\(^2\). Actually, the extreme wind speed may not coincide with the critical wind direction owing to the tracks of typhoons. Hence, the assumption could contribute to the overestimation of wave height and the consequent overtopping/overflow probability.

Thirdly, high wind speed and high water level was assumed to be fully correlated, but the example of Fung-wong showed that the assumption might not be in line with the reality.

In the light of the discussion, boundary conditions affected by typhoon are very complex. Different typhoon events may lead to entirely different conditions. In this section, the effects of typhoons’ features are only discussed conceptually. In a higher level of reliability analysis, the effects of typhoon should be incorporated into probabilistic calculation. A possible approach is to generate numerous random typhoon events statistically with diverse tracks and intensity. For each particular typhoon event, the marginal probabilistic distributions of the variables and dependence model among one another should be specified, and the reliability of the system can be calculated independently. The final results of the reliability can be derived by weighing the probability of occurrence of each typhoon event.

\(^2\)The wind direction that can lead to high probability of overtopping/overflow through high probability of occurrence, or long effective length, or small reduction factor.
Conclusions and Recommendations

The primary aim of this MSc study is to develop a method of estimating the reliability of the flood defences along the Huangpu River. The method contains two phases, namely proof of concept phase and specific analysis (overtopping/overflow). In each loop, subgoals were formulated:

- **Phase I: proof of concept**
  - Identify the more vital failure mechanisms within the stretch.
  - Identify the influential variables of the vital mechanisms that are worth further studying on.
  - Identify weak segments of the picked stretch.
- **Phase II: specific analysis**
  - Involve the effects of the variables into the method to improve the accuracy and reliability of the results.

Based on the results of the study, conclusions have been made in this chapter. Moreover, the reliability analysis method could still be improved if more loops were performed, so recommendations are proposed. In the end of this chapter, a reflection on the applicability of the VNK method in other cities are given.

### 8.1. Conclusions

#### 8.1.1. Reliability Analysis Method

- **A reliability analysis method for the flood defence system along the Huangpu River is developed.**
  The reliability analysis method has been developed on the basis of VNK2 project. The method contains multiple loops with which the results get updated and become more and more consistent with the reality. The key loop is the proof of concept loop where a full reliability analysis procedure

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1 Notice that in Chapter 6, some discussions and recommendations were also given, but they were made for the first loop only. The conclusions and recommendations in this chapter are made for the entire study based on the updated perceptions after both loops.
Conclusions and Recommendations

was performed. This loop involves most of the elements in a reliability analysis. In the end, it should provide the information about weak segments and influential failure modes. The quality of inputs regarding them significantly contribute to the accuracy of the results. Therefore, the following loops are based on it.

- **A model for probabilistic calculation is developed.**
  A tailor-made model has been built in Matlab. It is able to calculate conditional failure probability of single components under single water level condition and unconditional failure probability of the entire system. This version provides two computational methods, MCS and FORM. For successors furthering this study, the model is highly customisable, and it does not have strict requirements on the completeness of the input data.

8.1.2. Recognitions about the Selected Stretch

- **Closure failure of floodgates, overtopping/overflow are vital failure mechanisms.**
  Overtopping/overflow, piping, structural failure and closure failure of the flood gates were identified as main failure modes for the flood defence system along the Huangpu River. Among them, closure failure of floodgates is the most influential failure mode, followed by overtopping/overflow. The probability of structural failure is nearly close to 0. Piping also has large failure probability. However, due to the clay-dominated soil constitution, piping might not be a threat, which has to be further verified.

- **Section 1, 2 and Gate a are vulnerable to overtopping/overflow; Section 5-8 and Gate d, e are vulnerable to piping.**
  Section 1, 2 and Gate a are vulnerable to overtopping/overflow because the crest level of them is much lower than the adjacent; the Section 5-8 and Gate d, e are vulnerable to piping because the floodwall profile B (Figure 2.4b) is sensitive to the piping/seepage.

8.1.3. Overtopping/overflow

- **Distribution of extreme wind speed of typhoon in Shanghai has been derived**
  The extreme wind speed in Shanghai is approximately from 16m/s in 10-year condition to 22m/s in 10000-year condition. The estimation is much smaller than the initial assumption of 33m/s. The left-truncated Weibull distribution was suggested being the best-fit probability distribution to the extreme wind speed.

- **Effects of wind direction are incorporated in the method**
  The effects of wind direction are embodied in (a) probability of occurrence, (b) effective fetch, (c) the reduction factor in the limit state function. They were involved into the method and led to a drop by a factor of 4 in the overtopping/overflow probability.

- **Estimated overtopping/overflow probability has been less conservative but still high**
  The failure probability due to overtopping/overflow has been updated with a drop by a factor of 1000. However, it is still high, because the updated fragility curve 7.14 showed that some segments (e.g. Gate a) would be bound to fail due to overtopping/overflow given the 500-year water level, which is not in line with the 1000-year design water level for the flood defences along the lower reaches of the Huangpu River.

- **The reduction of wind speed contributes the most to the overtopping/overflow probability, but it is not prior to validate the data source of wind speed**
Wind speed is the most influential factor in the overtopping/overflow mechanism, so the data quality of it is rather important. However, after taking the estimated extreme wind speed, less conservative assumptions of wind speed distribution would not affect the results significantly. Hence, it is not urgent to improve the data quality of wind speed or validate the data source.

- **Dynamic features of typhoon can cause overestimation of the wind wave.**
  According to the example of typhoon Fung-wong, wind speed, wind direction, fetch, wave set-up can vary intensely and rapidly. The extreme wind speed might only last for tens of minutes during which the consequences by flooding may not be severe. Moreover, the dynamic feature of typhoon can cause phase difference of these factors. Therefore, the wind wave was still overestimated.

### 8.1.4. Other Conclusions

- **The failure probability of the 10km stretch is higher than expected, about 0.033-0.055 per year.**
  The failure probability of the stretch (1/30-1/20) is high. Also considering that there are at least 45 such stretches along the Huangpu River, the failure probability will be even higher. Therefore, the failure probability must have been overestimated.

- **The reliability analysis method can be applied in Shanghai. However, high data quality is required; model factors have to be calibrated; structural failure should be specified.**
  The VNK2 project and PC-Ring model do entail comprehensive hydrological and geotechnical information. Especially in an early stage to apply them to flood defence system without access to good data or without measurements of good data, the procedure in VNK2 cannot be fully taken. Besides, the limit state functions applied to vertical structures along the Huangpu River have to be modified by calibrating the model factors. Additionally, the limit state function of structural failure has not been provided particularly for the structures along the Huangpu River, so it should be specified.

### 8.2. Recommendations

In this study, a number of assumptions were made conservatively, which resulted in a conservative failure probability for the flood defence system. In this section, recommendations are given to improve the assumptions for the potential further loops.

Apart from academic recommendations for further study, practical recommendations for the authorities and other stakeholders are also given.

#### 8.2.1. Failure due to Non-closure of the Floodgates

Failure due to non-closure of the floodgates has become the most influential failure mode in the stretch, therefore, it is prior to make less conservative assumptions through further study.

1. **Assumption:** Failure due to non-closure was taken as a single event with a conditional failure probability of 1/100 per event.
   **Recommendation:** Failure due to non-closure should be taken as a series of events as is shown in Figure 8.1. Storage capacity and tolerance of the structures against collapse should be studied on and incorporated in the method.
2. **Assumption**: Closure of failure was assumed to be 1/100 per event, and it is homogeneous under all the water level condition.

**Recommendation**: It is recommended to investigate the difference of failure probability under different water level conditions. The "failure of closing process" might be a function of water level; "failure by inflow" is also a function of water level, because the storage capacity and tolerance of the structures against flooding depend on water level condition.

3. **Assumption**: 5 homogeneous floodgates were assumed to be present within the stretch.

**Recommendation**: An investigation should be done to collect the information about the exact number, the ownership, closing regime (e.g., automatically or manually) of floodgates.

4. **Assumption**: The closing process of the floodgates were mutually exclusive to one another.

**Recommendation**: Dependence of the floodgates should be estimated and incorporated, based on the investigations. When more than 30 floodgates were present within the stretch, the differences caused by involving dependence is significant.

![Figure 8.1: Recommended fault tree for failure due to non-closure of floodgates](image)

8.2.2. Piping

The failure probability by piping was calculated to rank the second in this study. However, it was also concluded that piping could hardly occur in the clay layer. So, further work is recommended to clarify the piping probability.

1. **Assumption**: A limit state function similar to Bligh was applied for probabilistic calculation

   **Recommendation**: The function was proved not to be suitable for probabilistic calculation, because the result is very sensitive to the factor "C". Hence, an advanced limit state function should be taken as a substitute.

2. **Assumption**: By the Huangpu River where there is thick clay layer, seepage could only happen along the contact surface between soil and structures

   **Recommendation**: The essences behind it should be studied on to verify whether the assumption is plausible, based on which limit state functions can be defined.
8.2.3. Overtopping/overflow

Overtopping/overflow probability is still high in spite of updating, so further work still has to be done but with lower priority.

1. **Assumption:** Model factors of wave height formula and limit state function of overtopping/overflow were simply quoted from the VNK2 project.
   **Recommendation:** Model factors have to be calibrated or a new formula has to be created in order to adapt to the situation of wave generation and structural characteristics along the Huangpu River.

2. **Assumption:** Extreme wind speed was assumed to be fully correlated with extreme water level.
   **Recommendation:** The correlation between the two extreme conditions can be studied on. In order to do so, detailed data about water level and wind speed are required.

3. **Assumption:** Critical Discharge was assumed to be a constant (10 l/m/s) referring to the flood-walls in New Orleans.
   **Recommendation:** The critical discharge should be redefined and should be taken as a stochastic variable. The backlands casted in concrete is stronger than grass or soil, and the storage capacity should be considered. Besides, duration of the extreme wind and wave condition should be involved, as it affects the amount of inflow. The critical discharge is expected to be higher than the initial assumption.

4. **Assumption:** The on-site wind speed was assumed to be equal to the measured wind speed.
   **Recommendation:** Effects of the distance and the high-rise buildings in the downtown on the wind speed can be studied on.

5. **Assumption:** The characteristics of typhoon was not considered into the method.
   **Recommendation:** The hydraulic boundary conditions specially under the influence of typhoon should be studied on in a probabilistic way.

8.2.4. Other Recommendations

1. **Assumption:** Elementary upper bounds and lower bounds were taken for combing the failure probability of the components.
   **Recommendation:** Correlation coefficients should be incorporated in the model.

2. **Assumption:** Only flooding condition (i.e., high water level) were taken into consideration in this study.
   **Recommendation:** Potential hazards in the normal water level conditions can be studied on. These threats include ship collisions, adjacent underground constructions, etc. Besides, failure mechanisms related to low water level conditions such as sliding of outer slope and structural collapse towards the river should be also incorporated into the method.

3. **Assumption:** The land subsidence and absolute sea level rise were constant values based on the difference of 2016 and the year of measurement.
   **Recommendation:** Land subsidence and absolute sea level rise can be taken as a function of time. It is expected that the reliability of the flood defences is dynamic, and with time, the failure probability can increase.
8.2.5. Recommendations to Stakeholders

1. Provided that the estimation is reliable, at the stretch $19km - 21km$ away from the river mouth, the crest level is lower than the adjacent floodwalls, and this stretch is the weak part of the subsystem. The backlands are threatened by overflow under the water level condition of 100 years and higher. Therefore, measures should be taken. For example, an option is to heighten the crest level.

2. The function to verify piping/seepage in the design stipulation [15] should be updated for the design of the flood defences.

3. The speed and direction of wind should be measured; the height and period of wind-induced wave should be modeled along the Huangpu River.

4. Flood risk analysis is recommended to perform by incorporating flooding scenarios and consequence analysis.

5. The effects on wind should be incorporated into the feasibility analysis of constructing high-rise buildings.

6. The reliability method should be also applied in other cities such as Tokyo and Jakarta, as the three cities are all under the threats of typhoon-induced floods. However, the method is recommended to be customised for different cities.

8.3. Reflection on the Applicability of VNK method

The reliability analysis method in the VNK2 project was applied to the flood defence system along the Huangpu River. In general, the method and the model can be applied in Shanghai and other cities, but some specific points have to be kept in mind.

Firstly, the actual circumstance and flooding situations of the city should be specified. For instance, important failure mechanisms should be investigated, according to the historical events and the characteristics of the local flood defences. Correspondingly, limit state functions should be customised for those failure modes. In different areas, the drivers of flooding can also be different, which depends on the climatic feature of the region.

Especially in developing cities (e.g. Shanghai, Jakarta, Ho Chi Minh City, etc.), data quality is limited, and it set barriers to the application. Take Shanghai for example, the information of typhoon and water level was only collected for less than 50 years. In some years, data are even largely missing. Therefore, the reliability of estimated extreme wind speed and the extreme water level can be doubted. Besides, because of the rapid development, the boundary conditions have been varying rapidly. In Shanghai, there has been four rounds of massive reinforcement and heightening of the flood defences, and the estimation of extreme water level has also been updated for three times [37]. Apart from it, the rapid construction of other infrastructures in the city also led to the changes of the boundary conditions. This is a common issue in other developing cities across the world as well. Because of this, the applicable data is even more limited; the historical events recorded could not be easily used to calibrate the estimated failure probability.

Finally, the tools used in this study can also be applied in other cities. For example, the PC-Ring model or the tailor-made model in this study can be used for the reliability analysis of a single segment or of the entire system. The Bayes model can be used for the extreme value analysis. The tool Fetch can be used to calculate the effective fetch length.
Deterministic Estimation

The reliability analysis focuses on the failure probability of the targets. Probabilistic calculation is therefore the main approach in this study. However, a deterministic calculation ahead of probabilistic simulation is useful. It gives an overview of how the subsequent probabilistic calculation could behave. Sometimes, for example, to evaluate whether a factor is worth studying and to save manpower and computational resources, deterministic calculations are executed. In this appendix, the procedures and relevant knowledge about it are reported.

A.1. Wave Height Approximation

A.1.1. Classical Formulas

In this thesis, the following three formulas were considered to estimate the wave height in the Huangpu River:

- **Formula from the China’s design norm**

  \[
  \frac{gH}{U^2} = 0.13 \tanh \left[ 0.7 \left( \frac{gd}{U^2} \right)^{0.7} \right] \tanh \left[ \frac{0.0018 \left( \frac{gF}{U^2} \right)^{0.45}}{0.13 \tanh \left[ 0.7 \left( \frac{gd}{U^2} \right)^{0.7} \right]} \right]
  \]

  in which
  
  - \( H \) = significant wave height (m)
  - \( d \) = average water depth (m)
  - \( U \) = wind speed at 10m high (m/s)
  - \( F \) = fetch (m)
  - \( g \) = gravitational acceleration (9.81 m/s\(^2\))

- **Bretschneider**

  \[
  H = \frac{0.283 u^2 v_1}{g} \tanh \left[ \frac{0.0125 \left( \frac{gF}{U^2} \right)^{0.42}}{v_1 \left( \frac{gF}{U^2} \right)^{0.75}} \right]
  \]

  \[
  v_1 = \tanh \left[ 0.530 \left( \frac{gF}{U^2} \right)^{0.75} \right]
  \]
A. Deterministic Estimation

in which

\[ H = \text{significant wave height (m)} \]
\[ d = \text{average water depth (m)} \]
\[ U = \text{wind speed at 10m high (m/s)} \]
\[ F = \text{fetch (m)} \]
\[ g = \text{gravitational acceleration (9.81 m/s}^2\) \]
\[ v_1 = \text{auxiliary variables, without physical meaning} \]

• Young & Verhagen

\[
\frac{gH^2}{16U^4} = 3.64 \cdot 10^{-3} \left( \tanh(A_1) \cdot \tanh \left[ \frac{B_1}{\tanh(A_1)} \right] \right)^{1.74}
\]

\[
A_1 = 0.493 \left( \frac{gd}{U^2} \right)^{0.75}
\]

\[
B_1 = 3.13 \cdot 10^{-3} \left( \frac{gF}{U^2} \right)^{0.57}
\]

in which

\[ H = \text{significant wave height (m)} \]
\[ d = \text{average water depth (m)} \]
\[ U = \text{wind speed at 10m high (m/s)} \]
\[ F = \text{fetch (m)} \]
\[ g = \text{gravitational acceleration (9.81 m/s}^2\) \]
\[ A_1, B_1 = \text{auxiliary variables, without physical meaning} \]

Usually, Bretschneider formula and Young & Verhagen formula are well utilised in the Dutch conditions. Specifically, the former is applicable in the case of broad space such as seas and oceans [41]. The latter formula is well applied for shallow lakes with short fetch [42]. Both formulae entail further calibration for particular riverine situation in Shanghai. By contrast, the first formula from the design norm was developed in such a certain case, which brings it priority to be applied.

In the case that better accuracy is required, the formulae have to be further calibrated by adding model factors. The factors can be determined by fitting additional experiments, which were however not dealt with in this thesis.

A.1.2. Effect of Wind Speed

The effects of varying wind speed on overflow/overtopping discharge is illustrated in Figure A.1. Crest level \( R = 0.2m, 0.4m, 0.6m, 0.8m \) approximately represent the water level condition of 2,000 years, 1,000 years, 500 years, 200 years in Section 8.

Take the 2,000-year water level condition \((R = 0.2m)\) and the 2,000-year wind speed condition \((U = 20m/s)\) for example. A change of 10% from the wind speed of 20m/s is shown in red, which leads to an approximately 25% increase for the discharge. A change of 5% can lead to only 12% of the discharge difference. When the crest height is larger or the wind speed is lower, the increase rate becomes larger. However, in general, the overflow/overtopping discharge is not very sensitive to the wind speed.
A.2. Structural Failure of Floodgates

In this study, the two limit state functions were customised to estimate the reliability of floodgates with respect to the stability of their beams and hinges respectively. To obtain an overview of the safety level, a deterministic calculation was performed. The results are introduced in this section.

A.2.1. Deterministic Estimation

To perform a deterministic calculation, the characteristic values were derived, based on the assumed mean values and standard deviations. The principles are introduced as follow:

- For the variables following the deterministic distribution, the characteristic values are equal to the deterministic values. For instance, $\rho_w = 1,000 \text{ kg/m}^3$.
- The rest of the variables were simply assumed to follow the normal distribution. Conservatively, the characteristic values of them were specified according to the following formula:

$$X = \mu \pm \sigma \cdot z_{\alpha}$$  \hspace{1cm} (A.1)

in which

$X$ = characteristic value
$\mu$ = mean value
$\sigma$ = standard deviation
$z_{\alpha}$ = a factor indicating the confidence level

$z_{\alpha}$ is equal to 1.645 given a confidence level of 90%. Besides, a plus sign is taken for "loading" variables, while a minus sign is taken for "resistance" variables.

Take the gate e for example, and the water level was assumed to reach the top level of the gate. The characteristic values are listed in Table A.1. The next step was to rewrite Formula 4.1 and 4.3, and it
delivers the results in Table A.2. Apparently, in terms of both beams and hinges, loading is dramatically smaller than resistance. This conclusion is proved by the ratio of S and R in the last column of the table. Therefore, from deterministic point of view, the floodgates are rather structurally solid, also considering the conservative assumptions made.

Table A.1: Characteristic values of variables: floodgate

<table>
<thead>
<tr>
<th>Variable</th>
<th>Unit</th>
<th>Distribution</th>
<th>Mean</th>
</tr>
</thead>
<tbody>
<tr>
<td>$z_0$</td>
<td>$m + MSL$</td>
<td>Normal</td>
<td>7.03</td>
</tr>
<tr>
<td>$z_s$</td>
<td>$m + MSL$</td>
<td>Normal</td>
<td>4.57</td>
</tr>
<tr>
<td>$\rho_w$</td>
<td>$kg/m^3$</td>
<td>Deterministic</td>
<td>1,000</td>
</tr>
<tr>
<td>$g$</td>
<td>$m/s^2$</td>
<td>Deterministic</td>
<td>9.81</td>
</tr>
<tr>
<td>$f_b$, $f_h$</td>
<td>$N/mm^2$</td>
<td>Normal</td>
<td>284</td>
</tr>
<tr>
<td>$h_{beam}$</td>
<td>$mm$</td>
<td>Normal</td>
<td>267</td>
</tr>
<tr>
<td>$l_{beam}$, $L_{gate}$</td>
<td>$mm$</td>
<td>Normal</td>
<td>4,165</td>
</tr>
<tr>
<td>$D$</td>
<td>$mm$</td>
<td>Normal</td>
<td>29.5</td>
</tr>
<tr>
<td>$n_b$</td>
<td>-</td>
<td>Deterministic</td>
<td>4</td>
</tr>
<tr>
<td>$n_h$</td>
<td>-</td>
<td>Deterministic</td>
<td>8</td>
</tr>
</tbody>
</table>

Table A.2: Results of deterministic estimation for structural failure of the floodgates

<table>
<thead>
<tr>
<th>Segment</th>
<th>$Z$</th>
<th>$R$</th>
<th>$S$</th>
<th>$S/R$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td>279.6$N/mm^2$</td>
<td>284$N/mm^2$</td>
<td>4.4$N/mm^2$</td>
<td>0.016</td>
</tr>
<tr>
<td>Hinge</td>
<td>190,249$N$</td>
<td>194,112$N$</td>
<td>3,863$N$</td>
<td>0.020</td>
</tr>
</tbody>
</table>

A.2.2. Sensitivity Analysis

Now comes a question: how solid the “solid” is of the floodgates. In order to visualise it, an approach is to vary some of the variables till the structure fails ($Z < 0$), and then to observe whether the value of each variable when failure could occur in reality.

Figure A.2: Sensitivity analysis of structural failure of floodgates. Relative value is the ratio of the value and the initial assumption.
A.2. Structural Failure of Floodgates

Horizontal Beams

Figure A.2a presents the variation of $Z$ with three varying variables. However, the rest of variables are not included, because in essence, the values of them were already assumed conservatively.

As is shown in the graph, only the decrease of $f_b$ could cause a significant drop of the $Z$ but with great "efforts". For instance, $Z$ falls below 0 only when relative $f_b$ decreases less than 0.02 and $f_b = 5.68 \, N/mm^2$. Apparently, this condition could hardly occur in reality.

Possibly, conservative conditions could appear simultaneously. Exaggerative assumptions were made and are listed in Table A.3. This time, $Z = 120N/mm^2$ and $S/R = 0.20$, which is still away from the failure.

Table A.3: Characteristic values of variables with exaggerative assumptions: beams

<table>
<thead>
<tr>
<th>Variable</th>
<th>Unit</th>
<th>Distribution</th>
<th>Mean</th>
</tr>
</thead>
<tbody>
<tr>
<td>$z_0$</td>
<td>$m + MSL$</td>
<td>Normal</td>
<td>8</td>
</tr>
<tr>
<td>$z_s$</td>
<td>$m + MSL$</td>
<td>Normal</td>
<td>4.57</td>
</tr>
<tr>
<td>$\rho_w$</td>
<td>$kg/m^3$</td>
<td>Deterministic</td>
<td>1,000</td>
</tr>
<tr>
<td>$g$</td>
<td>$m/s^2$</td>
<td>Deterministic</td>
<td>9.81</td>
</tr>
<tr>
<td>$f_b$</td>
<td>$N/mm^2$</td>
<td>Normal</td>
<td>150</td>
</tr>
<tr>
<td>$L_{beam}$</td>
<td>$mm$</td>
<td>Normal</td>
<td>8,000</td>
</tr>
<tr>
<td>$h_{beam}$</td>
<td>$mm$</td>
<td>Normal</td>
<td>267</td>
</tr>
</tbody>
</table>

Hinges and Bolts

Similarly, Figure A.2b presents the variation of $Z$ for hinges. Only the decrease of $f_h$ and $D$ could cause a significant drop. For instance, $Z$ falls below 0 when either relative $D$ drops to 0.18 ($D = 5.3mm$) or relative $f_h$ decreases to 0.02 ($f_h = 5.68N/mm^2$). Nevertheless, either condition could hardly occur independently in reality. Table A.4 is a list of extremely exaggerative assumptions. With them, failure could occur, $Z = -2,346N$ and $S/R = 1.09$. Again, the assumptions were too exaggerative to show up in reality.

Table A.4: Characteristic values of variables with exaggerative assumptions: hinges and bolts

<table>
<thead>
<tr>
<th>Variable</th>
<th>Unit</th>
<th>Distribution</th>
<th>Mean</th>
</tr>
</thead>
<tbody>
<tr>
<td>$z_0$</td>
<td>$m + MSL$</td>
<td>Normal</td>
<td>8</td>
</tr>
<tr>
<td>$z_s$</td>
<td>$m + MSL$</td>
<td>Normal</td>
<td>4.57</td>
</tr>
<tr>
<td>$\rho_w$</td>
<td>$kg/m^3$</td>
<td>Deterministic</td>
<td>1,000</td>
</tr>
<tr>
<td>$g$</td>
<td>$m/s^2$</td>
<td>Deterministic</td>
<td>9.81</td>
</tr>
<tr>
<td>$f_h$</td>
<td>$N/mm^2$</td>
<td>Normal</td>
<td>150</td>
</tr>
<tr>
<td>$L_{gate}$</td>
<td>$mm$</td>
<td>Normal</td>
<td>8,000</td>
</tr>
<tr>
<td>$D$</td>
<td>$mm$</td>
<td>Normal</td>
<td>15</td>
</tr>
<tr>
<td>$n_b$</td>
<td>-</td>
<td>Deterministic</td>
<td>4</td>
</tr>
<tr>
<td>$n_h$</td>
<td>-</td>
<td>Deterministic</td>
<td>4</td>
</tr>
</tbody>
</table>

A.2.3. Closure

In conclusion, the deterministic estimation showed that the structural failure of the floodgates could hardly happen. However, it is worth mentioning that, the conclusion was made given the limit state
functions 4.1 and 4.3. In practice, the formulas might not be appropriate to indicate the structural failure, because, for example, structural degradation was not involved. Therefore, the failure probability is not necessarily 0, despite both deterministic returned positive "Z".
Computational Methods

To perform a process of the reliability analysis in the first loop, a couple of computational models were correspondingly used. The essences behind them are introduced in this appendix.

**B.1. Unconditional Failure Probability**

The conditional failure probability only deals with the failure in the condition of a certain water level. In essence, it has to be translated into unconditional $P_f$ by applying the following formula:

$$P_f = P(f \mid z_{0l}) \cdot P(z_{0l})$$

**(B.1)**

in which
- $z_{0l} =$ water level $(m + MSL)$
- $P_f =$ unconditional failure probability when water level is $z_{0l}$
- $P(f \mid z_{0l}) =$ conditional failure probability given a water level of $z_{0l}$
- $P(z_{0l}) =$ probability of occurrence of $z_{0l}$

In each water level condition, there exists a $P_f$. However, the failure of a segment or a system for instance, could occur not only in the 10,000-year water level condition, but also in a water level lower or higher than the 10,000-year. Therefore, the overall failure probability in various water conditions can be formulised by summing up each $P_{f_l}$:

$$P_f = \sum_{l=1}^{n} P(f \mid z_{0l}) \cdot P(z_{0l})$$

**(B.2)**

For continuous probability functions, it becomes

$$P_f = \int_{0}^{\infty} P(f \mid z_{0})f(z_{0})dz_{0}$$

**(B.3)**

**B.2. Methods of Probabilistic Calculation**

As is mentioned, to carry out a reliability analysis, three levels of calculation are included in classification: Level I, Level II, and Level III. In the first loop, the Level III method, or specifically, Monte Carlo
simulation (MCS) method was mainly adopted, because the amounts of computation were still doable for a proof of concept method which has limited computational complexity.

**B.2.1. Monte Carlo Simulation**

MCS is a rather “clumsy” but effective method. By randomly generating billions of samples, computers imitate the billions of distinct stochastic events with billions of combinations of variables. Apparently, it is of no possibility to have such situations happen in reality. The number of stochastic samples is $N$. With the defined limit state functions: $Z = R - S$, the computer is able to filter the negative $Z$ and count the number of it, $N_f$. Given sufficient times of sampling, the ratio $N_f / N$ can represent the failure probability of the target.

**B.2.2. First Order Reliability Method**

The First Order Reliability Method (FORM) has proven to be highly efficient in the case of smooth limit state functions [43]. It is able to provide more information than MCS. For instance, it concludes an influence factor $\alpha$, which presents the influence of the corresponding variable in a limit state function. Moreover, it can also deliver a vector of design point, which is the cluster of values at which the failure is most likely to happen.
C.1. Probability Distributions for POT

In the Bayes Module, four probability distributions (i.e., exponential distribution, Generalised Pareto Distribution, Left-Truncated Weibull Distribution, and Pareto distribution) are candidates in order to perform a Peak-over-Threshold analysis (POT). In this appendix essences of them are introduced. Note that, since Pareto distribution and exponential distribution are two special cases of generalised Pareto distribution, they are not introduced specially in this section.

C.1.1. Left-truncated Weibull Distribution

The Left-truncated Weibull Distribution (LTWD) is a distribution transformed from the Weibull distribution. The two have the similar functions. The cumulative distribution and probability density functions of the (untruncated) Weibull distribution are shown respectively:

\[
F(x) = 1 - \exp(-\alpha x^\beta)
\]

\[
f(x) = \alpha \beta x^{(\beta-1)} \exp(-\alpha x^\beta)
\]

in which \(x > 0\), \(\alpha\) is a scale parameter, while \(\beta\) is a shape parameter, and \(\alpha > 0\), \(\beta > 0\). By adding a truncation point, the cumulative distribution function (CDF) and probability density function (PDF) of the LTWD are indicated as follow [44]:

\[
F(x) = 1 - \exp[-\alpha(x^\beta - T^\beta)]
\]

\[
f(x) = \alpha \beta x^{(\beta-1)} \exp[-\alpha(x^\beta - T^\beta)]
\]

where \(\alpha > 0\), \(\beta > 0\), and \(x > T > 0\). The additional \(T\) is the truncation point, a positive constant. LTWD becomes untruncated Weibull distribution as \(T = 0\). In the case of POT analysis, \(T\) is the threshold to prescribe for datasets.

A set of typical examples of LTWD is illustrated in Figure C.1. In the figure, \(\alpha = 1\) and \(\beta = 2\), while \(T\) varies from 0. When \(T = 0\), the solid line presents a Weibull distribution without truncation. It is also clear that, by prescribing thresholds in the other three curves, steps emerge. It resembles that the left parts of the graphs are cut out, and the remaining parts are statistically large values worth focussing on in the extreme value analysis.
C.1.2. Generalised Pareto Distribution

Another popular probability distribution is generalised Pareto distribution (GPD). The CDF and PDF of it are specified respectively as follow [45]:

\[
F(x) = \begin{cases} 
1 - \left(1 - k \frac{x - \xi}{\sigma}\right)^{1/k}, & k \neq 0 \\
1 - \exp\left(-\frac{x - \xi}{\sigma}\right), & k = 0 
\end{cases}
\]  
(C.5)

\[
f(x) = \begin{cases} 
\frac{1}{\sigma}\left(1 - k \frac{x - \xi}{\sigma}\right)^{1/k-1}, & k \neq 0 \\
\frac{1}{\sigma}\exp\left(-\frac{x - \xi}{\sigma}\right), & k = 0 
\end{cases}
\]  
(C.6)

in which, the scale parameter \(\sigma > 0\). \(\xi\) is a location parameter and \(k\) is a shape parameter. For \(k \leq 0\), the range is \(\xi \leq x < \infty\), while for \(k > 0\), \(\xi \leq x \leq \xi + \sigma/k\). The GPD reduces to the Pareto distribution when \(k < 0\) [45]:

\[
F(x) = \begin{cases} 
1 - \left(\frac{x_m}{x}\right)^a, & x \geq x_m \\
0, & x < x_m 
\end{cases}
\]  
(C.7)

\[
f(x) = \begin{cases} 
\frac{a\xi^a}{\sigma^a+1}, & x \geq x_m \\
0, & x < x_m 
\end{cases}
\]  
(C.8)

Moreover, GPD reduces to exponential distribution when \(\xi\) and \(k\) are both equivalent to zero:

\[
F(x) = \begin{cases} 
1 - e^{-\lambda x}, & x \geq 0 \\
0, & x < 0 
\end{cases}
\]  
(C.9)
Similar to LTWD, GPD also specifies a threshold whose information is contained in the location parameter $\xi$. As is shown in Figure C.2, for varying $\xi$, merely the location is shifted, unlike the case of LTWD. However, the similarity is that the focus is on the upper tail of the probability distribution, and the small values are excluded by setting the threshold.

$$f(x) = \begin{cases} \lambda e^{-\lambda x}, & x \leq 0 \\ 0, & x < 0 \end{cases}$$  \hfill (C.10)

Figure C.2: Example of generalised Paredo distribution ($k = 0.4, \sigma = 2$)

### C.2. Parameter Estimator

#### C.2.1. Maximum-likelihood Estimation

Maximum-likelihood Estimation (MLE) is a classical method aiming at estimating the parameters of statistical model by fitting the given data. This method was analyzed and vastly popularized by Ronald Fisher between 1912 and 1922 [46].

Suppose independent observations of $x_1, x_2, x_3, \ldots, x_n$, which reflecting the actual situation. In this study, they are the inputs of wind speed. $\theta$ is defined as a vector of parameters for the probability distribution. Moreover, in this method, a likelihood function can be specified:

$$L(\theta; x_1, \ldots, x_n) = f(x_1, \ldots, x_n \mid \theta) = \prod_{i=1}^{n} f(x_i \mid \theta)$$  \hfill (C.11)

It represents the joint likelihood of occurrence of the observations $x_1, x_2, x_3, \ldots, x_n$, so the maximum of $L(\theta; x_1, \ldots, x_n)$ expresses the most likelihood that $x_1, x_2, x_3, \ldots, x_n$ are observed simultaneously.
Therefore, the core idea of the MLE is to derive a $\theta$ vector that can lead to the maximum $L(\theta; x_1, ..., x_n)$. Usually, the goal can be achieved by differentiating with respect to $\theta$. Sometimes, this process could be complicated and could involve other numerical methods.

### C.2.2. Bayesian Parameter Estimation

This estimating method is built on Bayes’ theorem. The latter describes the probability of an event from an angle of conditional probability: the posterior event happens in the condition of the prior one. The theorem can be formalised as follow:

$$P(A \mid B) = \frac{P(B \mid A)P(A)}{P(B)} \tag{C.12}$$

where $A$ and $B$ are two stochastic events.

The theorem is applied to the estimation of probability distribution parameters. In contrast with the classical statistical methods, $\theta$, the vector of the parameters for the probability distributions, is a variable instead of a constant. The uncertainty directly results from a limited amount of observations. As a stochastic variable, $\theta$ also owns its own probability distribution. The uncertainty of $\theta$ directly determines the uncertainty of probability distribution of the extreme values.

Bayesian analysis defines two categories of uncertainty: uncertainty of prior distribution and posterior distribution. Now Formula C.12 can be rewritten into the following:

$$P(q \mid x) = \frac{l(x \mid q)P(q)}{P(x)} = \frac{l(x \mid q)P(q)}{\int l(x \mid q)P(q) dq} \tag{C.13}$$

in which $P(q)$ represents a prior density function, while $P(q \mid x)$ stands for a posterior density function. $x$ is a set of observations, and $l(x \mid q)$ is the likelihood function of observations given the prior distribution. With assumed prior distribution beforehand, posterior distributions and consequent parameters $\theta$ can be derived, based on the observations.

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\footnote{Most concepts in this section are quoted from Bayes Manual [31]}
Bibliography


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