Robustness quantification for navigation locks that are part of the primary flood defense

A study into the effect of increasing the reliability requirement of structural elements on the robustness of the structural system

D. Cornelissen
Robustness quantification for navigation locks that are part of the primary flood defense

A study into the effect of increasing the reliability requirement of structural elements on the robustness of the structural system

by

D. Cornelissen

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Thesis committee:
Prof. dr. ir. S. N. Jonkman
Ir. W. F. Molenaar
Prof. Dr. Ir. R. D. J. M. Steenbergen
Ir. M. Versluis

TU Delft, Hydraulic structures & Flood risk
TU Delft, Hydraulic structures & Flood risk
TU Delft, Structural Mechanics & TNO
Witteveen+Bos Consulting Engineers B.V.

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This thesis report marks the end of my master studies in Hydraulic Engineering at Delft University of Technology. It contains a resume of the research that I have worked on, in cooperation with Witteveen+Bos. They offered me the opportunity to study this topic by means of an internship at their office in Rotterdam. I am very grateful for having had this opportunity. I would like to thank my committee member at Witteveen+Bos, Marco Versluis. Throughout the whole graduation process he has been very committed and of great value to me. I would also like to thank all other colleagues at Witteveen+Bos for welcoming me to the company and sharing their knowledge.

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Summary

Introduction
In the Netherlands there are two laws that set requirements for the reliability ($\beta_{\text{design}}$) of structural elements: the Water Law and the Law on Buildings. An argument to prefer one law over the other, is the hypothesis that a higher reliability requirement results in a higher level of robustness of the structure. Therefore, the objective of this research is to quantify the effect of the reliability requirement for structural elements on robustness of the total structure.

For this research the scope of structures is narrowed down to navigation locks that are part of the primary flood defense.

Robustness
‘Robustness’ is a frequently used term, however engineers still struggle with the exact definition and how to include robustness in their designs. In 2011, the Joint Committee on Structural Safety (JCSS) has published a supporting document that gives a practical interpretation of the concept of robustness [16]. They refer to a publication by Baker, Schubert & Faber [2] in which a robustness index is proposed to quantify the level of robustness:

\[ I_{\text{rob}} = \frac{R_{\text{direct}}}{R_{\text{direct}} + R_{\text{indirect}}} \]

\[ R_{\text{direct}} = \sum P(H) \cdot P(D|H) \cdot C(D) \]

\[ R_{\text{indirect}} = \sum P(H) \cdot P(D|H) \cdot P(F|D) \cdot C(F) \]

The robustness index has a value between 0 and 1. According to this definition, a distinction has to be made between direct risk ($R_{\text{direct}}$) and indirect risk ($R_{\text{indirect}}$).

The direct risks are related to the initial damage of a structure (D). Indirect risks are related to subsequent failure (F) of the total structural system.

Damage and subsequent failure are considered to be caused by a hazard (H). Hazards are described as threats to the integrity of the structure. These are events that cause (accidental) loads on the structure. This is illustrated in Figure 1.

The basic steps in determining the (direct) risks of damage are:

1. $P(H)$: Identification of hazards and occurrence probabilities.
2. $P(D|H)$: Determination of the damage probability due to the considered hazards.
3. $C(D)$: Determination of the damage consequences.

Subsequently, the remaining steps in determining the (indirect) risks of failure are:

4. $P(F|D)$: Determination of the failure probability of the damaged system.
5. $C(F)$: Determination of the failure consequences.

Note that the robustness index that results from this framework, is heavily dependent on the definition of ‘damage’ and ‘failure’.

Case study
The effect of the reliability requirement on the robustness index is examined by means of a case study. For this the navigation lock at Weurt has been selected. The robustness indices are calculated for $2.00 \leq \beta_{\text{design}} \leq 6.00$ [yr$^{-1}$].

Hazards
High water levels and ship collision have been identified as the two most relevant hazards for water retaining structures. For both hazards the PDF of their loads is determined.

A negative correlation coefficient is assumed for multiple failures due to ship collision. For water
level differences it is assumed that failure events are positively correlated.

**Damage**

Based on a Failure Mode Effects & Criticality Analysis (FMECA), the lock gates have been found to be the most critical elements of the structural system. Damage is modeled as the sudden loss of a lock gate (Figure 2).

![Figure 2: Modelling of the damage state](image)

The resistance of the lock gate, is assumed to be equal to the resistance of the critical supporting beam. Subsequently, the resistance is modeled as a normally distributed variable.

The probability of damage is calculated by integrating over the product of the PDF of the load \( f_H(x) \) and the CDF of the resistance \( F_R(x) \):

\[
P(D \cap H) = \int_{-\infty}^{\infty} F_R(x) \cdot f_H(x) \, dx
\]

**Failure**

The navigation lock has multiple functions, each function requires its own failure definition. The main functions, flood defense and navigation, have been considered.

A fault tree analysis (FTA) has been performed to determine the probability of failure of the total structure. Within this approach, it is important to account for interdependencies between the structural elements (correlation).

**Results**

With respect to flooding, the robustness index is low for low values of the reliability requirement \( \beta_{\text{design}} \leq 4.25 \). See Figure 3. The low values are caused by the fact that the (indirect) risk of flooding is about 100 to 150 times larger than the (direct) risk of damage.

Because of the assumed correlations, the (indirect) risk of flooding is governed by the probability that all lock gates fail due to a water level difference.

Figure 4 presents a closer look on the robustness indices for \( \beta_{\text{design}} \leq 4.25 \). It shows that increasing \( \beta_{\text{design}} \), either has a negative effect or a positive effect on the robustness index. Increasing \( \beta_{\text{design}} \) not only reduces the (indirect) risk of system failure, but also the (direct) risk of damage. Whether robustness increases or decreases, depends on the relative reduction of the direct and indirect risk.

For stricter requirements, \( \beta_{\text{design}} \geq 4.50 \), the robustness suddenly increases. This is caused by the fact that ship collision becomes the governing load instead of water level differences. To satisfy the reliability requirement, the structural elements now have to be designed to withstand a ship collision. This means that the resistance has to be increased substantially (almost by a factor 10).

With the increased resistance, the probability that a lock gate fails due to water level differences is approximately zero. Because of the assumed correlations, the failure probability of the total system (multiple lock gates) also approaches zero: the (indirect) risk of flooding becomes negligible. The result is a high robustness index.

With respect to navigation delay, the robustness index is relatively high for all values of \( \beta_{\text{design}} \) (Fig-
This is caused by the fact that the consequences of navigation delay (indirect risks) are relatively small.

Again, at $\beta_{\text{design}} \geq 4.50$, a jump in the robustness index is recorded. Similar to the robustness for flooding, this is caused by the fact that ship collision becomes the governing load.

**Other measures**

In addition to increasing the reliability requirement, the effect of redundant critical elements is examined. In the case study, the redundant element is an additional lock gate. Figure 5, clearly shows that the additional lock gate results in a positive effect on the robustness index.

The benefit of redundant elements, is that they do not reduce the (direct) risk of damage. Only the (indirect) risk of system failure is reduced. Hence, the effect on the robustness index is always positive.

**Conclusions**

- Increasing the reliability requirement of individual structural elements $\beta_{\text{design}}$, does indeed effect the robustness of a structural system. However, whether the effect is positive or negative depends on the ratio of direct and indirect risks. In addition, the magnitude of the effect is limited.

  - A substantial increase of robustness, occurs when the (indirect) risk of system failure is significantly reduced. In the case study, this occurs due to a substantial increase of the resistance. This increased resistance, is not necessarily representative for other cases. Nonetheless, it shows that reduction of the (indirect) risk of system failure is an effective strategy to increase robustness. In essence, consequences of failure are fixed. Therefore, the main strategy to increase robustness is reduction of the system failure probability.

  - Based on the results from the case study, adding redundant elements to the design seems to have an advantage over increasing the reliability requirement. However, a statement about which measure is preferred, should be based on cost-benefit as well. With current knowledge, it is possible to calculate the required investments (€) for both measures. Though, the robustness index itself, is a dimensionless number which makes it hard to express the benefits in terms of money.

**Recommendations**

- Measures to increase robustness should focus on reducing the probability of system failure.

  - A method should be developed to express robustness in terms of money (€). At this point, robustness is a dimensionless number. Expressing robustness in terms of money will help to justify measures to improve robustness. For example, risks can be expressed in terms of money. This provides a valid argument to decide on what measures should be taken to control the risk.

**Reflection on the applied framework**

The advantage of the applied framework, is that it is easy to implement. It shows that a reduction of the risks of system failure is most effective to increase robustness. However, it doesn’t tell when the structure is sufficiently robust.

On the other hand, a risk-based approach does tell when the risk is sufficiently reduced. The defined allowable risk levels, are based on (i.a.) cost-benefit analysis. This is not yet possible for robustness, since the robustness index is dimensionless.

In addition, the main goal of a structure is to supply certain functions. One could argue that a structure should be considered robust, as long as these functions are guaranteed. This implies that a structure that satisfies (risk-based) reliability requirements, is a robust structure.
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Introduction and theory
1.1. Motivation

Robustness is a hot topic within the field of structural engineering. Not only structural systems are deemed to be robust. Also in other fields of engineering robustness is seen as an important characteristic of a system. However, practising engineers still struggle with the exact definition of ‘robustness’ and how to put it into practice.

The interest for robustness in this research originates from the introduction of new safety standards for the primary flood defence system in the Netherlands. These new standards are stipulated in the Water Law (Dutch: Waterwet) and set requirements for the reliability of primary flood defences. Hydraulic structures, that are a part of these flood defences, have to satisfy these reliability requirements as well. In addition, these hydraulic structures have to satisfy the reliability requirements that are set by the Law on Buildings (Dutch: Woningwet). Complicating fact is that both laws use a different approach to derive the reliability requirement. Hence, a conflict arises.

Within this discussion, which law to use, robustness is used as an argument. The general idea, is that a structural system is more robust when it satisfies a higher reliability requirement. This research examines this relation between the reliability requirement and the level of robustness of a structural system. The problem is illustrated in Figure 1.1.

![Diagram](image.png)

Figure 1.1: Illustration of the problem description
In simple words, the reliability requirement is no less but a maximum allowable probability of failure. In design practices, this maximum failure probability is applied on the individual elements of a structure. Whether or not the structural system fails, largely depends on this failure probability of the individual elements out of which it is constructed.

Many types of hydraulic structures exist, with different functions and different designs. From the spectrum of possible structures, this research focuses on one specific type of hydraulic structures: the navigation lock. Navigation locks are part of two important systems. The first system is the flood defense system. As such, navigation locks have to prevent flooding. The second system is the system of main waterways. As a part of this system, navigation locks have to allow passage of ships.

Recent events - Ship collision with the weir at Grave
Although the focus of this research is on navigation locks, a recent event has shown the relevance of robustness for another type of hydraulic structure. On December 29, a ship collided with the weir at Grave. The weir failed and as a result, the water level on the river Meuse dropped significantly and navigation on the river was obstructed. In addition, houseboats and yachts ran aground and reliability of the dikes might be undermined. Overall, the damage is enormous.

Now, the question arises whether this structure is (or was) sufficiently robust. An answer to this question will be given in the conclusions.

1.2. Research objective
From the introduction it has become clear that the concept of robustness is complex. However, the goal of this research is not to find a definition of robustness. Definitions and quantification methods for robustness are available from literature. As mentioned, the origin of interest in robustness lays in the discussion what reliability requirement should be used for hydraulic structures. For this research, the following objective is defined:

"The objective of this research is to quantify the effect of the reliability requirement for structural elements on the level of robustness of a navigation lock that is part of the primary flood defence system."

By achieving this objective, a statement can be made whether or not it is legitimate to use robustness as an argument to increase the reliability requirement. In addition, insight will be provided in the parameters that provide robustness of a system. These insights can be used to give recommendations for measures that possibly increase the robustness.

In order to achieve the objective, research question are formulated. A first set of questions is formulated to provide a better understanding of the concepts of reliability and robustness:

1. What methods are used to derive the structural reliability requirement?
2. How is robustness defined and what parameters are involved in quantification of robustness?
3. What is the relation between robustness and the reliability requirement?

The second set of questions focuses on the analysis of results that are produced by performing a case study. First of all, the aim of this set of questions is to analyse the effect of the reliability requirement on the level of robustness. In addition, the questions aim to identify other possible measures to increase robustness.

4. What is the effect of increasing the reliability requirement on the level of robustness?
5. What is the effect of adjusting other parameters that are involved in robustness quantification?
1.3. Approach and report structure

The approach of this research is in correspondence with the sequence of the research questions. First, more information about the concepts of reliability and robustness is collected. Subsequently, a case study is performed to analyse the effect of the reliability requirement on the level of robustness. To do so, the level of robustness will be calculated for multiple values of the reliability requirement.

Part I covers the first set of questions. First, a general description of navigation locks and their characteristics is given. This chapter is included to provide more insight on the general aspects of the design of navigation locks. The chapters that cover the concepts of reliability and robustness have a more theoretical character. These provide the framework that will be used in the case study.

Subsequently, Part II covers the case study that is performed. Chapter 5 presents an analysis of the systems of which the navigation lock is part of. It provides important information of the characteristics of the systems, such as loads and failure definitions. Subsequently, Chapter 6 goes into detail on the probability of damage. That is, the probability that an individual element of the structural system fails. The resulting damaged system is assessed in Chapter 7. This chapter provides the probability that the parent systems of the structure fail, given that damage has occurred. Finally, this part concludes with Chapter 8 that presents the resulting robustness indexes of the structure for both parent systems.

The report concludes with the conclusions and recommendations in Part III. In consecutive order the research questions are provided with an answer. Recommendations are given to improve accuracy of the results and for further research.

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**Figure 1.2: Structure of the report**
In total there are 1,777 hydraulic structures in the Netherlands with a water retaining function in the Netherlands (primary and secondary) [4]. There are numerous types of hydraulic structures, this research focuses on one specific type: the navigation lock.

This chapter starts with a general description of navigation locks, their functions and their parent systems (2.1). Subsequently, the general lay-out of a lock complex and design requirements are discussed in Section 2.2. Finally, legislation of lock design is discussed in Section 2.3. This section reveals the actual conflict between the Law on Buildings and the Water law.

2.1. General description

Navigation locks are generally part of an overarching lock complex with multiple navigation locks. In general, a navigation lock is a system that is composed of structural, mechanical and electrotechnical elements. Structural elements are (mainly) visible for users. Mechanical and electrotechnical elements are not. This makes that the design and control of a lock complex is multidisciplinary and not always easy to comprehend. The focus of this research is on the structural elements. The lay-out and individual elements of navigation locks are discussed in Section 2.2.

A special characteristic of navigation locks is that they are part of multiple parent systems. Hence, they are designed to fulfill multiple functions. The main functions always are its flood defense function and the navigation function. Additional functions are possible as well. Examples of additional functions are water management, passage of road traffic and cultural-historical value. The more functions, the more complex the design process becomes. This research only focuses on the main functions of navigation lock, these are discussed separately in Sections 2.1.1 and 2.1.2.

2.1.1. Primary flood defence system

A large part of the Netherlands is situated below mean sea level. This causes that the flood defence system is of critical importance to protect economical, natural and human value. In addition to the threat by the sea, many rivers that run through the country introduce the risk of flooding as well.

In order to prevent the country from flooding, an extensive system of flood defenses has been constructed with a total length of approximately 3,800 kilometres. Only a small part of the country (about 3%), is situated in so called "unembanked" areas. Well known examples of flood defenses are dikes, dunes, dams, barriers and other hydraulic structures. The focus of this research is on navigation locks that are part of the primary flood defenses.
Primary flood defenses can be roughly subdivided into three types, see Figure 2.1. Type A primary flood defenses directly retain high waters, preventing the hinterland from flooding by rivers, lakes or sea. Type B primary flood defenses connect two dike rings and have a primary function to retain water. Finally, type C primary flood defenses are defenses that only become active when another primary flood defense fails. Whether a flood defence is primary type C or regional, depends on its national importance. From this description it seems obvious that navigation locks are generally not located in a type C flood defense.

Dike rings
The primary flood defence system is built-up out of dike rings, in total there are 95 dike rings. In Appendix A an overview of the dike rings in the Netherlands is presented. Starting from 2017 each dike ring trajectory is given a maximum allowable flooding probability. Based on this requirement the elements of the dike ring are designed to have a sufficient reliability. How the reliability requirement is determined and how this is incorporated in the design is explained Chapter 3.

2.1.2. Main waterway system
The many rivers that run through the Netherlands not only cause a risk of flooding. These same rivers make that the Netherlands is a perfect logistical hub that connects Europe with the rest of the world. For this reason, the main waterway system in the Netherlands is intensively used by all types of vessels. The total length of the Dutch main waterway system is approximately 1.400 kilometers, see Figure 2.2.

Vessels that make use of the inland waterways have various dimensions up to a length of 147.15 meter (in coastal areas larger vessels may navigate). See Appendix A for a complete overview of ship dimensions. Vessels require a minimum draught which cannot always be guaranteed due to natural variations in the river discharge. For this reason, many waterways have weirs that make sure that there is sufficient water depth. Passage for navigation is then provided by navigation locks.

Whereas low discharges cause problems for navigation, exactly the opposite applies for the flood defense system. For the flood defence system it are the high discharges that cause problems. Due to discharge variations and the intensive use by navigation improvements of the system are continuously conducted. These vary from dredging operations to more modern projects such as the ‘room for the river’-project.
2.2. General lay-out of navigation locks

Based on functionality, a lock complex can be subdivided into roughly three subsystems. These are the lock approach, the lead-in jetties and the navigation lock itself. All three subsystems are illustrated in Figure 2.3. The lock approach serves as an area where approaching vessels decrease their speed and get in queue for the lockage procedure. It provides mooring facilities for (at least) short periods of time. In some cases, mooring facilities allow for a longer stay. The lead-in jetties are the transition between the lock approach and the navigation lock. These jetties prevent collision of a vessel with the lock heads and guide the vessels into the lock chamber. In addition, visual and audible elements are present for instructions.

The navigation lock itself, can be subdivided into several elements as well. For the case study, an extensive subdivision of the navigation lock will be made (see Chapter 5). Here, only the main elements are highlighted. There are 3 main structural elements that make up the navigation lock: lock heads, lock gates and lock chamber. These are illustrated in Figure 2.4. In addition to the structural elements, mechanical and electrotechnical installations are an extremely important part of the navigation lock.

Lock heads have to supply sufficient strength and overall stability of the structure. Moreover, they have to provide a storage for the lock gates when the gates are in opened position. Generally, the lock gates do not have a role in the overall stability of the structure. They do however play a major role in retaining high water level. Therefore they should be designed with sufficient strength. Finally, the lock chamber has to provide sufficient overall stability and strength as well. Forces on the chamber wall mainly originate from the soil body, water pressures and navigation.
General costs of a navigation lock can be subdivided into investment costs and lifetime costs. Investments are costs that relate to the design and construction. Lifetime costs consist of maintenance, repairs and demolition costs. An estimation of the total costs of Sluis Eefde is presented in Figure 2.5. From this calculation it becomes clear that, in general, the investment costs are governed by the structural elements. Lifetime costs on the other hand are large for mechanical and electrotechnical installations.

![Figure 2.5: General costs of a navigation lock (example: Eefde) [35]](image)

2.3. Legislation & guidelines

In the Netherlands two separate laws influence the design of hydraulic structures: the Water Law [24] and the Law on (residential) Buildings [23]. Hydraulic structures have to satisfy the different safety requirements from both laws. The laws are discussed separately in Sections 2.3.1 and 2.3.2.

Over the past decades, guidelines have been developed that translate these requirements into a sufficient design. Examples are the 'Leidraad Kunstwerken' [32], the 'Richtlijnen Ontwerpen Kunstwerken' [29] and the book "Ontwerpen van Schutsluizen" [9]. These guidelines are not further elaborated.

2.3.1. Water Law

The Water Law was introduced in 2009 and is the result of a recollection of 8 former laws. It sets out the rules and responsibilities for management of Dutch water bodies and safety requirements for the flood defense system. Reliability requirements for hydraulic structures are derived in the guideline 'Leidraad Kunstwerken' (LK). These are based on the exceedance probability of a specific water level.

Through the Deltaprogram, changes have been made to the Waterwet. These became effective on January 1st, 2017. These changes include a change of safety approach: instead of designing for a specific water level exceedance probability, the design will be based on a probability of flooding. The allowable probability of flooding is based on the expected consequences. As a result of this change in safety approach, new guidelines have been developed for the design and assessment of flood defenses.

The new guideline for assessments, WBI2017 [6], gives a detailed description of the derivation of the new safety requirements. These safety requirements, however, are not yet incorporated for hydraulic structures in the new (temporary) design guideline, the OI2014 [28]. In the long run, the OI2014 has to provide a basis for a new guideline that replaces the current 'Leidraad Kunstwerken'. Reliability requirements for hydraulic structures should then be based on the new approach that considers the probability of flooding.

2.3.2. Law on Buildings

The Law on Buildings states that structures in the Netherlands should satisfy reliability requirements from the Bouwbesluit (English: "Building Agreement"). This is a collection of regulations that apply on all structures
in the Netherlands: both hydraulic structures and other structures such as housing, offices etc.

In 2012 the Bouwbesluit was updated. One of the most important changes was that the old standards, TGB1990 series, were replaced by the Eurocodes. Law on Buildings, Bouwbesluit en Eurocodes are used interchangeably in this report.

Reliability requirements are prescribed in the Eurocodes for three different consequence classes. As a result, the exact nature or function of the structure is of secondary importance. When consequences of failure are large, the structure should have a high reliability. For example, even though their functions are different, a football stadium and a governmental building should satisfy the same reliability requirement. This differs a lot from the Water Law which considers the specific flood defense function.

The Eurocodes have been developed for buildings and bridges. For that reason they are not perfectly suited to use for design of hydraulic structures. Nonetheless they should be complied. Some design challenges that are typical for hydraulic engineering, such as hydraulic loads, are completely missing in the Eurocodes. This is one of the reasons that guidelines such as the LK have been developed to get some grip on the design process.

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<tr>
<td>Eurocode: reliability index</td>
<td>Eurocode: reliability index</td>
</tr>
<tr>
<td>Dunes</td>
<td>Dunes</td>
</tr>
<tr>
<td>Dikes</td>
<td>Dikes</td>
</tr>
<tr>
<td>Structures</td>
<td>Structures</td>
</tr>
<tr>
<td>Unacceptable inflow</td>
<td>Failure of closure</td>
</tr>
<tr>
<td>Structural failure</td>
<td>Piping</td>
</tr>
<tr>
<td>Failure of closure</td>
<td>Structural failure</td>
</tr>
<tr>
<td>Overflow</td>
<td></td>
</tr>
</tbody>
</table>

Figure 2.6: Schematic overview of changes induced by the Deltaprogram. *In the new situation, the failure modes 'overflow' and 'overtopping' are not mentioned separately for structures. Because of large correlation these failure modes are combined for dikes and structures and only mentioned as failure modes for dikes. In design, these failure modes still have to be accounted for.*

### 2.4. Summary & conclusion

Since both laws use a different approach to derive the reliability requirements, their results are different. The question in this research is whether or not robustness can be used as an argument to pick one of two options.

Focus of the research will be on the structural elements of navigation locks that contribute to their two main functions: flood defense and navigation. Moreover, focus is on the application of the reliability requirement in the design stadium. This stadium is governing with respect to the costs of structural elements.
Structural reliability

A structural reliability requirement can be interpreted as a maximum allowable probability of structural failure. Or in other words the required probability of non-failure of a structure. Within structural engineering this reliability requirement is often expressed as a reliability index $\beta$. This chapter explains two methods to derive the reliability index and discusses the differences between these two methods.

The chapter starts with a description in Section 3.1 of possible failure modes. Subsequently in Section 3.2 the reliability index and its background are explained. Section 3.3 and Section 3.4 explain how the reliability index is derived respectively for the Water Law and the Law on Buildings. Then, in Section 3.5 the role of time dependence is explained. Finally, the chapter is summarised in Section 3.6.

### 3.1. Failure modes

Failure refers to the disability of a system to fulfill a specific function. Here, failure is considered as failure of the flood defense system: a flood occurs. Failure of the system can be caused by numerous initial events, these are named failure modes. Roughly 4 failure modes are distinguished for hydraulic structures: overtopping (and overflow), non-closure, piping and structural failure. See Figure 3.1.

![Fault tree of flooding, including initial events](image-url)
Overflow & overtopping
Overflow occurs when the still water level (i.e. without waves or the average level considering waves) is higher than the top of the structure and the water flows into the protected area. The overflow discharge volume can lead to flooding [19]. In contrast to overflow, with wave overtopping the still water level is below the crest level. The overtopping is purely due to waves breaking over the structures. The limit state for wave overflow and overtopping is usually defined in terms of critical discharges, which themselves depend on the available storage capacity of the protected water system [19].

Non-closure
definition: Non-closure doesn't necessarily lead to flooding. This depends, amongst other factors, on the water level of the outside water (river, lake or sea) and the storage capacity of the protected water system. In addition, it depends on the reliability of the bed protection. For situations with sufficient storage capacity, high velocities of the inflow might result in damage to the bed protection and subsequently result into erosion and failure of the structure's stability. Many events can lead to non-closure such as human failure, accidental loads and mechanical failure. The wide range of scenarios makes it hard to determine probability of failure due to non-closure. This failure mode is not relevant to all structures but it certainly is for navigation locks.

Piping
When hydraulic gradients in the subsoil towards the protected side are sufficiently high, soil particles start eroding, leading to the formation of cavities or channels in the subsoil - the so-called pipes. These pipes can grow to the high water-side of the structure, undermining its foundation, which can lead to collapse or sliding of the structure [19]. In the Netherlands piping is associated with the Dutch term “onderloopsheid”, which is really related to the development of pipes underneath the structure. In addition, there is “achterloopsheid” which is related to development of pipes around the structure.

Structural failure
This indicates failure of the structure itself, either as a whole or of specific parts of the structure. The whole structure fails when the equilibrium of forces (horizontal, vertical, moments) is lost. Specific parts of the structure fail when the strength of the element is not sufficient to bear the loads. This can either be the result of poor design/construction or an accidental load that was not accounted for in the design. Focus of this research is on this failure mode.

3.2. Reliability index
Reliability of a system can be expressed in terms of a ’β’. This section explains the meaning of this β and its relation to the probability of failure. First, the general concept of failure probability of a structural element is explained based on the joint probability of its resistance (R) and a certain load (S). Subsequently, in section 3.2.2 the mathematical relation between failure probability and reliability is explained.

For more information on the reliability index, it is referred to the TU Delft lecture notes on probabilistic design [20], the Eurocodes [25] and the Leidraad Kunstwerken [32].

3.2.1. Failure probability
The probability of failure of a structural element, is the probability that an occurring load exceeds the strength of the structural element. Thus, in order to calculate the failure probability, the design value of the load (S) and the design value of the strength (R) have to be known. In addition, it is required to have information
about the uncertainties. Information about the expected value and uncertainties of a variable are expressed by its probability density function.

Failure probabilities can also be explained with the help of a limit state function \((Z)\), see Formula 3.1. A limit state function basically is the subtraction of the load \((S)\) from the strength of the structure \((R)\). When the limit state function becomes smaller than zero, the structural element fails. In that case, the load is larger than the resistance. Similarly, the structural element survives as long as the limit state is larger than zero. In the case that both load and strength have deterministic values, the failure probability is either 1 or 0.

\[
Z = R - S
\]

\(Z\) = Limit state function
\(R\) = Strength (French: Resistance)
\(S\) = Loads (French: Solicitation)

When the probability density functions of \(R\) and \(S\) are known, these can be used to determine the probability density function of \(Z\). This is illustrated in Figure 3.3. Here, only the basics of the limit state function are discussed. For more information about the derivation of its probability density function it is referred to other literature [20].

In Figure 3.3 a Gaussian distribution is assumed for both the strength and the load. This is one of the most common distribution types. However, not all events have a probability density function that can be modeled using a Gaussian distribution. Other examples of well known continuous distributions are the log-normal distribution and the Gumbel distribution which are generally used for extreme values.

### 3.2.2. Mathematical relation

In Figure 3.4 a Gaussian distribution is assumed for both the strength and the load. This is one of the most common distribution types. However, not all events have a probability density function that can be modeled using a Gaussian distribution. Other examples of well known continuous distributions are the log-normal distribution and the Gumbel distribution which are generally used for extreme values.

![Figure 3.4: Left: calculation of the failure probability. Right: relation between reliability index \(\beta\) and failure probability.](image-url)
Failure probability and reliability are directly related through the standard normal distribution, see Formula 3.2. As one would expect, the reliability increases when the failure probability decreases. This works the other way around as well. The relation is illustrated in Figure 3.4.

\[
P_f = \Phi(-\beta)
\]

\[
\beta = -\Phi^{-1}(P_f)
\]

\[
P_f = \text{Failure probability} \\
\Phi = \text{Standard normal distribution} \\
\beta = \text{Reliability index}
\]

3.2.3. Calculation of failure probabilities

There are multiple ways for calculating the failure probability. For this research, the failure probabilities are calculated analytically. This calculation requires the cumulative distribution function of the resistance and the probability density function of the load. This is illustrated in Figure 3.5.

\[
P_f = \int f_R(M) \cdot f_M(M) \, dM
\]

\[
P_f = \int f_R(M) \cdot [1 - F_R(M)] \, dM
\]

Figure 3.5: Illustration of how failure probability and reliability are related through the standard normal distribution.

3.3. Structural reliability in the Water Law

This section starts with description of recent developments regarding structural reliability requirements from the Water Law. A transition has been made towards a flood risk approach. The procedure to derive reliability requirements within this approach is not commonly applied in current practice. Therefore, the procedure for deriving the reliability requirement is explained step-by-step.

3.3.1. Recent developments

Until recently, the procedure to determine reliability requirement for hydraulic structures was taken from the Leidraad Kunstwerken (LK) [32]. On January 1st 2017, the Water Law has been updated. As a result, the information in the Leidraad Kunstwerken is not fully in correspondence with law anymore.

In the new situation, the reliability requirements are based on a flood risk approach. Procedures for determining the reliability requirement are stipulated in documents that are part of the Wettelijk Beoordelingsinstrumentarium 2017 (WBI2017) [6, 7]. These documents are used for the assessment of existing flood defenses.

In addition to the WBI2017, the Ontwerp-Instrumentarium 2014 (OI2014) [28] has been developed for the design of flood defenses. As such, this document is the (temporary) successor of the LK. In the long run, a new version of the LK will be published. Nonetheless, for most failure modes of hydraulic structures the OI2014...
refers to the LK. Hence, the LK still has significant value in the design process. With respect to structural failure, the OI2014 refers to the Eurocodes instead of the LK. This has been the motive of this research.

Motivation of this research
With respect to structural reliability, the OI2014 states that it is to be expected that structures will satisfy the reliability requirement when they are designed using CC3 from the Eurocodes. Consequence class 3 sets a reliability requirement which is relatively high ($\beta_{design,1yr} = 5.20$), this could cause over-dimensioning of the structural elements. The possibility of over-dimensioning is (partly) justified by the idea that it results in a more robust structure.

3.3.2. Maximum allowable flooding probability

The maximum allowable flooding probability is derived per dike ring trajectory, based on an acceptable risk of flooding. When the consequences are known, the allowable probability can be calculated using the following definition of risk:

"Risk is the probability of an undesired event multiplied by the consequences" [20]

Consequences of flooding have been studied in the project 'Veiligheid Nederland in Kaart 2’ (VKN2) [36]. Accurate estimates of casualties and economic damage have been made. Hence, consequences of flooding are known. The acceptable risk of flooding is a political decision. The choice of the acceptable risk is, of course, substantiated. Important criteria are given in the report 'De betekenis van de norm' [18]:

1. Maximum local individual risk $\leq 10^{-5}$ (basic safety)
2. Results of social cost-benefit analysis
3. Adjustments based on group risk, vital infrastructure and governmental wishes (economic)

Based on the acceptable risk of flooding and the estimated consequences, a norm class has been set for each dike ring trajectory. In WBI2017 a total of seven norm classes is distinguished:

- 1:300 per year
- 1:1,000 per year
- 1:3,000 per year
- 1:10,000 per year
- 1:30,000 per year
- 1:100,000 per year
- 1:1,000,000 per year

These norm classes are based on conservative assumptions during their derivation. Therefore, when calculating the maximum allowable probability of flooding, the norm classes have to be multiplied by a factor $\zeta_{max}$ [18]. See Formula 3.3.

\[
(3.3) \quad P_{max} = \zeta_{max} \cdot P_{norm}
\]

- $P_{max} = \text{Maximum allowable flood probability}$
- $\zeta_{max} = \text{Factor to account for conservative assumptions}$
- $P_{norm} = \text{Norm specification flood probability}$

(in most cases: 3 for dike trajectories, 1 for dune trajectories)
3.3.3. Failure probability budget

The allowable flood probability from Section 3.3.2, can be used to determine the requirement for each element that is part of the considered dike trajectory. Examples of elements are dikes, dunes and structures. An illustration is given in Figure 3.6.

![Figure 3.6: Illustration of a dike ring trajectory](image)

The requirement for each element of the trajectory is determined using the failure probability budget that is given in Table 3.1. The allowable failure probability of the trajectory is equal the ‘total budget’ and is distributed over the elements. The WBI2017 uses a standardized budget for all dike ring trajectories [17].

Table 3.1 shows that for each element, multiple failure modes are considered to contribute to the total failure probability. Each failure mode gets a certain percentage of the total failure probability, this is the so called failure space $\omega$.

The advantage of using the same budget for each trajectory, is that only one set of partial factors has to be derived for semi-probabilistic calculations. The budget is based on the data that was obtained through fieldwork of VNK2 [36]. Hence, it has a good match with the current situation of the Netherlands. When desired, it is possible to use other budget distributions. However, this would require a fully probabilistic assessment.

![Table 3.1: Failure probability space (\(\omega\)) per failure mode [17]. *Governing load and strength parameters for this failure mode have such a large correlation in space that the failure probability space is combined for hydraulic structures and dike sections with equal orientation.](image)

<table>
<thead>
<tr>
<th>Structure</th>
<th>Failure mode</th>
<th>Sandy coast</th>
<th>Other (dikes)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dike</td>
<td>Overflow and overtopping*</td>
<td>0.00</td>
<td>0.24</td>
</tr>
<tr>
<td></td>
<td>Uplift and piping</td>
<td>0.00</td>
<td>0.24</td>
</tr>
<tr>
<td></td>
<td>Macrostability inner side</td>
<td>0.00</td>
<td>0.04</td>
</tr>
<tr>
<td></td>
<td>Damage revetment and erosion</td>
<td>0.00</td>
<td>0.10</td>
</tr>
<tr>
<td>Structure</td>
<td>Failure of closure</td>
<td>0.00</td>
<td>0.04</td>
</tr>
<tr>
<td></td>
<td>Piping</td>
<td>0.00</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>Structural failure</td>
<td>0.00</td>
<td>0.02</td>
</tr>
<tr>
<td>Dune</td>
<td>Erosion</td>
<td>0.70</td>
<td>0.00 / 0.10</td>
</tr>
<tr>
<td>Other</td>
<td></td>
<td>0.30</td>
<td>0.30 / 0.20</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td><strong>1.0</strong></td>
<td><strong>1.0</strong></td>
</tr>
</tbody>
</table>
3.3.4. Failure probability per failure mode

The focus of this research is on the reliability requirement for structural failure. Therefore, the failure probability budget (Table 3.1) and Formula 3.4 should be used to calculate the requirement for this failure mode.

\[
P_{\text{f,mode}} = \frac{P_{\text{max}} \cdot \omega_{\text{mode}}}{N}
\]

- \(P_{\text{f,mode}}\): Maximum probability of the considered failure mode
- \(P_{\text{max}}\): Maximum allowable flood probability of dike trajectory (section 3.3.2)
- \(\omega_{\text{mode}}\): Failure probability space for structural failure (section 3.3.3)
- \(N\): Length effect factor

Formula 3.4 contains one more parameter that needs explanation: the length effect factor 'N'. In a practical sense, this factor is used to account for uncertainties. This especially important for (continuous) series systems. A dike ring and its trajectory are a perfect example of a series system: failure probability of the system is governed by the failure probability of its weakest element.

Example: length effect for dikes

A dike is a continuous series system. A breach at any location, will result in flooding. Considering geotechnical failure (e.g. piping), soil characteristics play an important role.

Uncertainties in the soil characteristics are large due to the limited number of cone penetration tests. Hence, for increasing length of the dike, the probability that there is a weak spot in the soil layers increases. For this specific example, the length effect 'N' can be calculated with the formula below [28].

\[
N = 1 + a \cdot \left[ \frac{L}{b} \right]
\]

- \(N\): Length effect factor
- \(a\): Part of the trajectory that is sensible to the failure mode
- \(L\): Total length of the trajectory
- \(b\): Measure of length that represents the magnitude of the length effect within 'a'

The hydraulic structures that are part of a trajectory, together can be modeled as a (non-continuous) series system. The WBI2017 and OI2014 [6, 28] do not give a standard formula for the failure modes of hydraulic structures. Instead, a range of possible values is recommended including a brief substantiation.

**Structural failure, \(N = 1-10\)**

With respect to structural failure, the length effect depends on the number of structures. The available budget of failure probabilities has to be distributed over the number of structures. Therefore, a first indication is to assume that \(N\) is equal to the number of structures with a maximum of 10.

On the other hand, failure of only 1 structure is sufficient to cause flooding. In practice, the probability of joint failure of multiple structures should subtracted from the sum of failure probabilities of the individual structures. Hence, the first indication above is conservative.
3.3.5. Example calculation

By combining formula 3.4 with formula 3.2, it is possible to determine the reliability indices for structural failure that correspond to the norm class of the Water Law. For example, a navigation lock that has to be designed for a dike ring trajectory with norm class 1:10,000. First the maximum allowable flood probability is calculated using Formula 3.3:

\[ P_{\text{max, 1:10.000}} \approx 3 \cdot 1 : 10.000 = 1 : 3.000 \]

The objective is to calculate the reliability index for structural failure. So first the allowable failure probability is calculated. Failure space for structural failure is \[ \omega = 0.02 \] (Table 3.1). There are 2 structures in the trajectory, as a result \[ N = 2 \]. The allowable probability of structural failure for the navigation lock is calculated with Formula 3.4:

\[ P_{\text{f, structural}} = \frac{1 : 3.000 \cdot 0.02}{2} \approx 3.00 \cdot 10^{-6} \]

Using Formula 3.2, the corresponding reliability index can be calculated.

\[ \beta = -\Phi^{-1}(3.00 \cdot 10^{-6}) \approx 4.53 \]

3.4. Structural reliability in the Law on Buildings

Reliability indices that satisfy the Law on Buildings are stipulated in the Eurocodes [25]. The Eurocodes have made their entrance in the Netherlands in 2012 when they were included in the Bouwbesluit, replacing the old TGB1990 series.

In contrast to the Water Law the Eurocodes do not present the derivation method for reliability index. Instead it is qualitatively described what reliability index should be used for the design. Determining what reliability index should be used is based on so called consequence classes. The Eurocodes distinguish between three consequence classes. An overview of the consequence classes is presented in Table 3.2.

<table>
<thead>
<tr>
<th>Class</th>
<th>( \beta_1 )</th>
<th>( \beta_{50} )</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>CC3</td>
<td>5.20</td>
<td>4.30</td>
<td>Large consequences with respect to loss of life, or significant economic impact, social impact or impact on environment.</td>
</tr>
<tr>
<td>CC2</td>
<td>4.70</td>
<td>3.80</td>
<td>Medium consequences with respect to loss of life, or considerable economic impact, social impact or impact on environment.</td>
</tr>
<tr>
<td>CC1</td>
<td>4.20</td>
<td>3.30</td>
<td>Small consequences with respect to loss of life, or small economic impact, social impact or impact on environment.</td>
</tr>
</tbody>
</table>

In practice, hydraulic structures that are part of the primary flood defense are almost always classified as a CC3. As explained in 2.3.2, hydraulic structures are not mentioned separately in the Eurocodes. Instead they can be classified as ‘Other structures’. As a result, determination methods for hydraulic loads are not mentioned specifically in the Eurocode. Therefore often reference is made to the Leidraad Kunstwerken [32], “Richtlijnen Ontwerpen Kunstwerpen” [29] or the book “Ontwerp van schutsluizen” [9].

Although only the derivation method of the reliability indices in Table 3.2 is presented in the Eurocodes and not the derivation of the exact values, it is known that these are derived using a cost-benefit analysis over a certain reference period. Another contrast with the Water Law is given by the fact that the reliability indices are derived at the level of structural elements, and not at the level of a structural system.

It should be noted that the Eurocode considers the reliability indices only as a directive. It is allowed to deviate under the condition that this is very well substantiated, using a risk based approach.
Based on the values in the tables it can be seen that a lower reliability is required for larger reference periods. This can be defended from an economic perspective. Because of the fact that the structure lasts longer it generates more benefits. As a result the relative investments are lower and therefore a higher failure probability is accepted.

In the following section it is explained how these 1 year probabilities can be transformed into probabilities over a specific reference period.

### 3.4.1. Example calculation

To show the relation between reliability index and failure probability again, an example is given. Consider a navigation lock that is part of a 1:10.000 norm class. Subsequently the navigation lock will be classified as a CC3 structure. According to Table 3.2 the corresponding reliability index for a reference period of 1 year is equal to 5.20. Using Formula 3.2, this results in:

\[ P_f = \Phi(-5.20) \approx 9.96 \times 10^{-8} \]

Compared to the calculation in Section 3.3.5 the calculated failure probability is lower. This already illustrates the difference between Eurocode and Waterwet safety requirements. Based on legislation the most stringent requirement has to be applied, in this case the Eurocode.

### 3.5. Time dependence

Up to now, failure probabilities have only been considered with respect to a reference period of only 1 year. In practice, navigation locks are designed to have a lifetime of 100 years. Lock gates, mechanical- and electrotechnical installations are often susceptible to wear and have smaller design life times in the order of 10, 30 or sometimes 50 years.

Because both strength and loads may vary over time, it is rather difficult to calculate the exact failure probability over a specific period. This section discusses time dependent behaviour models, such as the bathtub curve. Subsequently, it is discussed how the Water Law and the Eurocodes use different models to account for time dependent behaviour.

#### 3.5.1. Bathtub curve

The lifetime and its individual years can be modeled as a series system. Each individual year can be interpreted as an element with a specific strength \( R_i \) and specific loading \( S_i \). Failure probability is the probability that \( R_i \) is smaller than \( S_i \). Characteristic feature of a series system is that failure of a single element leads to total system failure: the system is as strong as its weakest element. This holds as well for the lifetime and its individual years: failure in one year means that the structure fails within its lifetime.

![Bathtub curve diagram](image)

Failure of the system is easily calculated. Consider a structure with a design lifetime of 50 years \((n = 50)\). The individual years \((i = 1, \ldots, n)\) represent the elements each having a characteristic strength \( R_i \) and load \( S_i \). The probability that the system fails is equal to the probability that one of the elements fails:

\[(3.5) \quad P( \text{failure in lifetime} ) = P( \text{failure year 1} \cup \text{failure year 2} \cup \ldots \cup \text{failure year n} )\]

\[ P( \text{failure in lifetime} ) = P( R_1 < S_1 \cup R_2 < S_2 \cup \ldots \cup R_n < S_n ) \]
The difference in reference period between Law on Buildings and the WBI is not unique. This difference was already present in the situation with the LK \[32\]. Formula 3.6 is proposed in both the LK and the Eurocodes \[25\] to transform the failure probability. This formula assumes that the failure probabilities of the individual years are identical and independent. In reality the failure probabilities of the individual year are not independent and they are also not identical. By default, Formula 3.6 is therefore incorrect for exact calculations.

\[
P_{f,n} = 1 - (1 - P_{f,1})^n
\]

\(P_{f,n}\) = Probability of failure for reference period of ‘n’ years
\(P_{f,1}\) = Probability of failure for reference period of 1 year
\(n\) = Number of years within the reference period

Incorrectness of Formula 3.6 is explained by the fact that it neglects two important phenomena: (1) dependency between the individual years and (2) physical processes that influence both strength and load. Dependency between the individual years is caused by conditionality of failure.

Physical processes that play a role in hydraulic structures design are deterioration and load increase (e.g. due to climate change). Their expected values are expected to change unfavourable, resulting in a higher failure probability. Physical processes become more important when the structure is nearing its end of lifetime.

In the middle phase of the lifetime both processes are not significantly noticeable. As a result there are roughly three phases within the design lifetime. Plotting these phases results in a bath tub curve (Figure 3.8a).

Phase 1 Failure probability decreases as a result of ‘proven strength’
Phase 2 Failure probability becomes constant, calamities and extreme circumstances are dominant
Phase 3 Failure probability increases due to climate change and deterioration

The bath tub curve shows the conditional rate of failure \(r(t)\). The conditional failure rate gives the conditional probability of the failure density function at time \(\tau\), given that failure has not yet occurred at time \(\tau\).

\[
r(t)dt = P(R(\tau) < S(\tau) \text{ for } \tau \in (t, t + dt)) | R(\tau) > S(\tau) \text{ for } \tau \in (0, t))
\]

It is important to understand that this bath tub curve gives failure probabilities of individual years. The total failure probability within the design lifetime can be calculated by integration over the bath tub curve with respect to time. Time dependent behaviour can be related to the failure probability using Formula 3.8. When the functions of \(\Delta R(t)\) and \(\Delta S(t)\) are known, the exact failure probability can be calculated.
3.5. Time dependence

\[ Z = (R(0) - \Delta R(t)) - (S(0) + \Delta S(t)) \]

- \( R(0) \) = Initial strength (or resistance)
- \( S(0) \) = Initial loads (or solicitation)
- \( \Delta R(t) \) = Decrease of strength in time
- \( \Delta S(t) \) = Increase of loads in time

The accuracy of the failure probability function with respect to time strongly depends on the knowledge of the physical processes. Models are based on research and observations that often cover only a limited period of time. As a result, tools such as curve fitting and extrapolation are required for longer time spans. These introduce uncertainties into the model [21].

Other, more simplified, models are possible as well. Figure 3.8b for example neglects the effect of deterioration and climate change, resulting in a failure probability that only decays in time. In theory, at infinite time, the failure probability will become zero. Figure 3.8c also neglects these effects and in addition it neglects the phenomenon that a structure proves itself. In other words the failure probability in an individual year is assumed to be constant and independent from previous years.

![Figure 3.9: Effect of truncating the reference period at 10 years. The graph is a plot of Formula 3.6 with unrealistic high failure probabilities for the individual years. It is purely for clarification.](image)

The Leidraad Kunstwerken states that Formula 3.6 can be used for a maximum of 10 years. Even when the design lifetime is equal to 50 or 100 years. This is done to account for the above described time dependent phenomena, without the necessity to make an exact calculation [32]. By truncating the reference period at 10 years, a larger reliability can be guaranteed within the design lifetime. After all, Formula 3.6 in theory results in certain failure for large reference periods and/or large failure probabilities for the individual years. This is illustrated in Figure 3.9.
3.6. Summary & conclusion

**Summary**
Section 3.1 has explained the role of structural reliability within the design of hydraulic structures. Structural failure is one of the failure modes that contributes to the failure probability of a structure. Subsequently it was shown that the reliability index is directly related to the failure probability through Formula 3.9.

\[
P_f = \Phi(-\beta)\]

\[
\beta = -\Phi^{-1}(P_f)
\]

In Sections 3.3 and 3.4 derivation of \(\beta_{\text{design}}\) is explained for the two applicable laws. Reliability indices from the Water Law are derived at the level of the structural system but can be directly applied on the individual structural elements as well. Reliability indices from the Law on Buildings also apply at the level of structural elements.

Theoretical background of the derivation method is similar for both laws. Both use, nevertheless in a different way, a cost-benefit optimisation. The Water Law defines the consequences very accurately, based on results of the VNK2 studies [36]. Downside is that only consequences of flooding are considered. The Eurocodes define the consequences less accurately. An advantage is that consequences of all functions are included.

Finally, section 3.5 explains the influence of time dependent processes on the failure probability. Reliability indices from the Water Law have to be transformed for longer reference periods than the standard 1 year. Time dependencies make that this is a complex process. Therefore, it is decided not to account for time dependence in this research. Failure probabilities and reliability requirements will be used for a reference period of 1 year.

**Conclusion**
Reliability requirements from both laws, that correspond to a reference period of 1 year, are in the range between 3.00 and 5.00. This research considers \(2.00 < \beta_{\text{design}} < 6.00\). This will give good insight on the effect of the design value of the reliability index for structural elements.

Concluding, these values are used as input for robustness calculations. Taking a larger design value of the reliability index results in a lower failure probability of structural elements. By comparing results for different design values of the reliability index it can be examined how the level of robustness of a structure reacts.
Robustness

A higher design value of the reliability index is assumed to result in a more ‘robust’ design. The goal of this chapter is to find a definition of robustness and to provide a method that can be used to quantify the level of robustness of a system. Results of this chapter are used to examine whether the assumed relation between reliability index and robustness holds.

4.1. Historical perspective

Robustness has not always been part of building regulations. The first time it became relevant to building regulations was after the collapse of the Ronan Point apartment building in London in 1968. Due to a gas explosion on the 18th floor, adjoining structural elements progressively collapsed. The initial event of the gas explosion directly caused structural failure of the 18th floor and indirectly caused structural failure of all 23 floors. In other words, the initial event led to relatively large consequences.

Another example of progressive collapse is the failure of the World Trade Center in New York after the terrorist attacks on 9/11. In the aftermath of this disaster the interest in robustness increased. Not only the improbable number of fatalities and the extend of damage played a role. Also other factors such as the symbolic nature of the buildings, motives of the attack and real-time television transmission had an important role in the impact of this event. [3]. Again, interest in robustness was renewed by an event that had extremely large consequences.
4.2. Definition of robustness

The examples of Ronan Point and the WTC already imply a certain definition of robustness. Both examples relate to relatively large and unexpected consequences but have differences in cause and scale. Importance of robustness is widely acknowledged but how should it be exactly defined?

4.2.1. Common language

Robustness, or robust, descends from the Latin words 'robus' and 'robustus'. Many translations are possible such as 'strong', 'firm', 'healthy', 'tough' and 'vigorous'. What these words have in common is that they provide a certain basis to overcome adverse conditions. Hence, in day-to-day life robustness of a system or organization is generally associated with the ability to withstand or overcome adverse conditions.

Another interpretation in common language is that a robust object has an above average strength. Normal objects with average strength have the ability to survive under normal conditions. In addition, robust objects may also survive under unforeseen circumstances. Thus in common language 'robustness' indicates the ability to overcome unforeseen adverse conditions.

4.2.2. Engineering practice

Engineers often encounter the requirement of a 'robust design' in project contracts. Robustness is mentioned in building codes and regulations however in practice different interpretations are possible. Building codes and regulations will be discussed later. This section explains interpretations from engineering practice which originate from conversations with professionals within the field of hydraulic engineering in the Netherlands (Witteveen+Bos and Rijkswaterstaat).

Robustness in practice is sometimes interpreted as meeting the standard: a prior prescribed maximum probability of failure. Other engineers interpret robustness to be meeting the standard plus a safety margin or additional performance beyond design load. In reality this comes down to a lower failure probability than prescribed which can be achieved in several ways.

Engineers often relate robustness to other criteria as well, such as endurance and low maintenance solutions. These criteria do not directly result in a lower probability of failure, but often do so indirectly. In the case of low maintenance the structure can be designed such that it is easy to reach and replace elements susceptible to failure. So what happens in reality is that the probability of timely recovery is increased, indirectly decreasing the probability of failure. In general, it is concluded that engineers feel that robustness is an additional part of the design process that mainly depends on experience, logic and common sense.

4.2.3. Building codes and regulations

A large overlap exists between codes and regulations on the one hand and engineering practice on the other hand about interpretations of the definition of robustness. This originates from the fact that engineers largely rely on specific design rules that are presented in codes, guidelines and regulations such as the Eurocode.

At present the Eurocodes are the governing document on structural design. These provide some information about robust design, but they don't give a definition that leads to a general requirement. Still the engineer has the responsibility to decide whether a design is robust or not. The Eurocodes define robustness in the document on accidental loads, NEN-EN 1991-1-7, as follows:

"Robustness is the ability of a structure to withstand events like fire, explosions, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause." [25]

Consequences should thus be related to an initial event and it should be verified whether they are disproportional. Severe consequences should not be the result of an event that is very likely to occur. But at what point does the consequence become disproportional to the cause? This question remains unanswered in the
4.2. Definition of robustness

Eurocodes and it is up to the engineer to answer it. In paragraph 3.2 of NEN-EN 1991-1-7 there are however suggestions given on how to include robustness in the design:

- **Strategy 1**: Determine on which elements the structure relies for its overall stability and design these elements as critical elements to increase the probability of survival.
- **Strategy 2**: Use materials with high ductility for the design of structural elements to create a high absorption capacity of deformation energy without causing cracks.
- **Strategy 3**: Provide sufficient redundancy to ensure that loads can be absorbed in alternative ways after occurrence of an accidental load.

It is important to note that, in the Eurocodes definition, robustness is not equal to resistance to accidental loads. All structures should be robust, regardless the likelihood of accidental loads.

### 4.2.4. Scientific papers

Although the Eurocodes definition gives a good understanding of the concept of robustness, it has the disadvantage that it doesn’t provide a quantifiable design criteria. For that reason many attempts have been made to find a suitable definition for robustness which can also be expressed in mathematical terms.

Early attempts were made in 1987 by Frangopol and Curly [8]. They proposed a method to determine the magnitude of the robustness related concept of redundancy. Their method was based on the difference in reliability between a fully operational stage and damaged state. In 1995 another study was performed by Lind [22] who proposed a definition of the vulnerability of a structure. He defined vulnerability as a measure to determine the loss of system reliability due to damage. As well vulnerability can be related to robustness.

In 2006 Baker et al. [2] proposed a quantification method, specifically targeting robustness. Their approach has been adopted in several other documents including documents of the Joint Committee on Structural Safety (JCSS). Their definition of a robust system is:

> "A robust system is one where indirect risks do not contribute significantly to the total system risk."

According to Baker et. al. [2] the corresponding index of robustness is:

\[
I_{\text{rob}} = \frac{R_{\text{direct}}}{R_{\text{direct}} + R_{\text{indirect}}}
\]

This definition makes a distinction between direct and indirect risks. Direct risks are related to initial damage caused by the load, leading to a reduced performance of the system. Indirect risks are related to subsequent failure of the system. Both types of risk are further explained in Section 4.3.

'System' refers to both a physical structure as well as its associated inspection, maintenance and repair procedures. The effect of inspection and maintenance is shortly elaborated in Section 4.3.5. However, this research focuses on only the navigation lock (physical structure) itself.

A robust structure, with negligible indirect risks, corresponds to an index equal to 1. The other extreme is a value of 0 which corresponds to a structure that is not robust at all: all events of failure will lead to failure of the system. By distinguishing direct and indirect risks the Eurocodes approach of disproportional consequences is included. In addition, it provides a basis for quantification of the robustness of a structural system.

### 4.2.5. Conclusion on the definition

From all four described perspectives robustness relates to the ability of a system to overcome adverse conditions without disproportional consequences. This disproportionality is thus a key element in the definition of robustness. What consequences should be considered acceptable and what is the turning point at which the
consequences become disproportional is not yet determined. Keeping in mind, that a definition is required that allows for quantification of the level of robustness the following definition is used:

"Robustness is the ability of a structure to withstand extreme events or the consequences of human error, without being damaged to an extent disproportionate to the original cause"

Within this definition extreme events include both identified and unidentified (accidental) loads. Robustness according to this definition can be quantified by quantifying the proportionality between cause and consequences. This proportionality will be quantified according the JCSS definition of the robustness index that follows from Formula 4.1.

4.3. Quantification of robustness

Following from Formula 4.1, direct and indirect risk are the key parameters to quantify robustness. Therefore robustness quantification comes down to quantification of the risk. First, a definition of risk is required in order to find proper values for the robustness index. Risk can be defined as:

Risk is a set of scenarios \( S_i \) each of which has a probability \( P_i \) and a consequence \( C_i \) [20]

The research aims to assess robustness with respect to failure, which indicates that the risk consists of a set of failure scenarios. Failure refers to not being able to execute (one of) the desired functions. These functions have been explained for navigation locks in Section 2.1. Failure scenarios consist of all possible (sets of) events that result in failure. Probability of these events and corresponding consequences can often be calculated or estimated. Based on Baker et al. [2], a set of failure scenarios can be graphically presented with the help of an event tree. See Figure 4.2.

![Figure 4.2: Robustness: event tree for robustness quantification (Baker et al. [2])](image)

Each branch of the event tree represents a failure scenario. The initial event is a specific hazard (denoted as \( H \)) which indicates that the system is exposed to a load case (e.g. ship collision, extreme water level) that has the potential to cause damage to the system. If no damage occurs (\( D \)), the scenario is complete. If damage occurs, a variety of damage states (\( D \)) is possible.

The damaged system either fails (\( F \)) or survives (\( F \)). Whether damage to the system results in failure of the system depends on the severity of the damage. One could imagine that only some scratches will not have a significant impact on the system’s strength. On the other hand, when damage means that structural components of the system fail, the system’s strength may be significantly reduced.

Consequences are related to possible states of damage and failure. Distinction is made between direct consequences (\( C_{Dir} \)) and indirect consequences (\( C_{Ind} \)). This formulation is in line with guidelines for risk assessment that have been developed by the Joint Committee on Structural Safety (JCSS) [16].
Figure 4.2 displays a total of three scenarios. Theoretically the event tree includes an infinite set of failure scenarios through the wide range of values possible for $H$ and $D$. These wide ranges are also illustrated in Figure 4.2. In general design procedures, this infinite set has to be converted into a finite set of scenarios.

### 4.3.1. Hazards

In the context of robustness, hazards are defined by the JCSS as follows:

"a hazard is a serious threat to the integrity of a structure and the safety of people." [16]

In other words, hazards are threats or events that may cause failure of a structure. In practice, this means that either an (additional) load is generated or that the resistance is compromised. Examples of hazards for navigation locks are high water levels, ship collision, deterioration, etc.

Hazards may have either natural or human origins. In some cases there is some overlap when human interventions result in a 'natural' hazard. An example of this is the occurrence of earthquakes in the northern part of the Netherlands caused by (human) gas extraction. Based on the JCSS report [16] hazards can be roughly subdivided into three categories:

1. **Nature and general human activities.** Natural hazards are extreme conditions of nature such as earthquakes, storm surges and tornadoes. Amongst others, explosions are an example of (unintentional) man-made hazards. For structural design the difference is hardly relevant.

2. **Deliberate actions.** Mostly man-made actions such as vandalism and malicious attacks (terrorism). For this type of hazards it is not always effective to design a stronger structure: the action is likely to be increased accordingly. Damage mitigating designs can be more efficient.

3. **Errors and negligence.** Examples of this hazard are design and construction errors, negligence of deterioration or operator errors. This type of hazards can be managed best be accurate quality control and supervision. At the same time they can be created by bad supervision and quality control.

### 4.3.2. Direct risks - System damage

Direct risks are those associated with the damage state ($D$) of the system. For navigation locks, this means failure of a structural component such as a lock gate or a culvert gate. Hence, direct consequences are those costs that have to be made to restore the structural components of the system. Direct risks can be calculated with Formula 4.2:

\[
R_{dir} = \sum P(H) \cdot P(D | H) \cdot C(D)
\]

- $P(H)$ = Probability that hazard $H$ occurs
- $P(D | H)$ = Probability that damage $D$ occurs, given the fact that hazard $H$ has occurred
- $H$ = Hazard
- $D$ = Damage of the system
- $C(D)$ = (Direct) Consequences of damage

### 4.3.3. Indirect risks - System failure

Indirect risks on the other hand, are those associated with subsequent failure of the system. Failure refers to loss of function(s) of the system. For navigation locks, this means flooding or significant delays for navigation.

Note that Formula 4.3 does not include the consequences of damage $C_{dir}$. Including these consequences...
4. Robustness

(4.3) \[ R_{\text{ind}} = \sum P(H) \cdot P(D|H) \cdot P(F|D) \cdot C(F) \]

\[
\begin{align*}
P(H) & = \text{Probability that hazard } H \text{ occurs} \\
P(D|H) & = \text{Probability that damage } D \text{ occurs, given the fact that hazard } H \text{ has occurred} \\
P(F|D) & = \text{Probability that system failure } F \text{ occurs, given the fact that damage } D \text{ has occurred} \\
\end{align*}
\]

\[
\begin{align*}
H & = \text{Hazard} \\
D & = \text{Damage of the system} \\
F & = \text{Failure of the system} \\
C(F) & = \text{(Indirect) Consequences of system failure} \\
\end{align*}
\]

would result in a distorted value of the robustness index. Figure 4.3 illustrates the difference between hazards, direct consequences and indirect consequences.

Based on a quick analysis of Formula 4.1, it is already possible to make a general statement about the effect of the reliability requirement on the level of robustness. When the change of the indirect risk is smaller than the change of the direct risk, the robustness is increased. Principal design strategies are discussed in 4.4.

The reliability requirement of a structural element (\(\beta_{\text{design}}\)) is related to the robustness index because it determines the probability of failure of the structural element. For the hazard 'H' that causes the governing load on the structural element, it can be stated that:

(4.4) \[ \Phi(-\beta_{\text{design}}) = P(D \cap H) = P(H) \cdot P(D|H) \]

The occurrence probability of the hazard 'P(H)' is not influenced by \(\beta_{\text{design}}\). Hence, the reliability requirement influences the robustness through the term \(P(D|H)\) as highlighted in Formula 4.4.

The reliability requirement \(\beta_{\text{design}}\) applies on all structural elements. Once damage occurs (e.g. one of the structural elements fails), the other structural elements determine the probability of failure of the remaining structure. Hence, robustness and the reliability requirement of structural elements are related through the highlighted parameters in Formula 4.5.

(4.5) \[
\begin{align*}
R_{\text{dir}} &= \sum P(H) \cdot P(D|H) \cdot C(D) \\
R_{\text{ind}} &= \sum P(H) \cdot P(D|H) \cdot P(F|D) \cdot C(F)
\end{align*}
\]
4.3.4. Example robustness quantification

The concept of robustness is quite abstract. For that reason it will be clarified by means of an example. The example considers a navigation lock that is part of a flood defense and connects a canal to a river. When both lock gates fail to retain high water levels on the river, the system fails and flooding of the city occurs.

First step in robustness quantification is that hazards have to be identified. For transparency only one hazard (H) is considered in this example. The considered hazard is an extremely high water level on the river, causing a critical water level difference over lock gate 1. Probability of occurrence is taken equal to:

$$P(H) = 10^{-5} \text{ yr}^{-1}$$

**Direct consequences** are those related to damage of the system. In this example, damage is interpreted as structural failure of gate 1. The resulting direct consequence is that a new lock gate has to be installed. The following characteristics apply:

$$P(D|H) = 10^{-2} \quad \text{(failure of gate 1, given the fact that the hazard occurs)}$$

$$C(D) = €1 \cdot 10^6 \quad \text{(costs to install new lock gate)}$$

**Indirect consequences** are those related to failure of the (damaged) system. In this example the indirect consequence is flooding of the city. The probability that flooding occurs is equal to the probability that gate 1 and gate 2 fail, given the fact that the hazard has occurred. The following characteristics apply:

$$P(F|D) = 10^{-1} \quad \text{(system failure: failure of gate 2, given the fact that gate 1 fails)}$$

$$C(F) = €1 \cdot 10^7 \quad \text{(damage due to flooding of the city)}$$

Using the above information, the event tree can be constructed, see Figure 4.5. Note that this example only serves to clarify the concept of robustness. A more realistic representation can be obtained by expanding the number of hazards and damage states and by improving the values of the probabilities and consequences.

![Event Tree](image-url)
**Direct risks** can now be calculated using Formula 4.2:

\[ R_{\text{dir}} = P(H) \cdot P(D|H) \cdot C(D) \]

\[ R_{\text{dir}} = 10^{-5} \cdot 10^{-2} \cdot 1 \cdot 10^6 \]

\[ R_{\text{dir}} = €0.10 \]

**Indirect risks** can now be calculated using Formula 4.3:

\[ R_{\text{ind}} = P(H) \cdot P(D|H) \cdot P(F|D) \cdot C(F) \]

\[ R_{\text{ind}} = 10^{-5} \cdot 10^{-2} \cdot 10^{-1} \cdot 1 \cdot 10^7 \]

\[ R_{\text{ind}} = €0.10 \]

The **robustness index** corresponding to these values can now be calculated using Formula 4.1:

\[ I_{\text{rob}} = \frac{R_{\text{dir}}}{R_{\text{dir}} + R_{\text{ind}}} \]

\[ I_{\text{rob}} = \frac{€0.10}{€0.10 + €0.10} \]

\[ I_{\text{rob}} = 0.50 \]

The robustness of the structure is now quantified for the considered conditions. In Section 4.4 it is shown how the robustness index reacts when its parameters are adjusted.

### 4.3.5. Extended model

The event tree of Figure 4.2 is an extremely simplified representation of reality. The model can be extended by adding branches to the tree. Assuming that the first hazard \((H_1)\) hasn’t resulted in failure but did cause damage \((D)\), the structure will have an increased vulnerability. Response actions \((a_r)\) can be taken to restore the initial resistance of the structure. Whether response actions are taken depends on whether the damage is detected \((I)\) or not \((\bar{I})\). The probability that a second hazard \((H_2)\) will result in failure of the damaged system depends on timely detection \((I)\) and subsequent measures \((a_r)\) that are taken. This is represented in the expanded event tree of 4.6.

![Figure 4.6: Robustness: event tree including inspection and response actions for robustness quantification [2]](image-url)

Making a robustness calculation using the extended model requires a lot more computing capacity and information about the failure probabilities. Considering navigation locks it is assumed that damage to the structural elements is detected and repaired in time. For that reason the extended model will not be used in this research.
4.4. Principal design strategies

In Section 4.2.3 already some design principles were mentioned. These design principles originate from robustness definition as it is given in the Eurocodes. More general design strategies can be derived from Formula 4.1. The interest is in the parameters that can be optimised in the design process. For this reason, first the hazard (H) is removed. Formula 4.1 then reduces to:

\[
I_{rob} \approx \frac{\sum P(D) \cdot C(D)}{\sum P(D) \cdot C(D) + \sum P(D) \cdot P(F|D) \cdot C(F)}
\]

Taking the initial exposure (H) out of the equation cannot be justified on a theoretical basis since the probabilities of system damage and subsequent system failure depend on it. In addition, when more than one hazard is considered, the probability of occurrence of a hazard largely determines the magnitude of its contribution to the robustness index.

However, from a more practical perspective it does provide good insight on the effect of adjusting the design parameters. Formula 4.6 contains a total of four parameters, see also Figure 4.7. Adjustments to these four parameters are identified as the principal design strategies to increase robustness.

The effectiveness of the strategies is now analysed, based on the input parameters from the example in Section 4.3.4. For each parameter the initial value is adjusted while other parameters are kept constant. The results are shown in Figure 4.8. Note that the outcomes are based on Formula 4.6 and therefore are only valid for robustness indices that consider a single hazard. In the case where multiple hazards have to be considered, the largest effect will be caused by the biggest contributors.

Based on Figure 4.8 it can be concluded that, for this specific system, it is not very effective to make adjustments to the damage probability. According to the graph, reduction of the parameters C(D) and P(F|D) proves to be more effective.

The effectiveness of the strategies largely depends on the initial value of the robustness index, this is shown in Figure 4.9. The graph shows the relative change of the robustness index when the probability P(F|D) is
increased or decreased. Multiple curves are plotted, each curve represents another initial value of the robustness index. As it appears, the robustness of systems with a high initial value are hardly affected. On the other hand, the robustness of systems with a low initial do seem to be sensitive to changes of the conditional probability of system failure \( P(F|D) \)

![Effectiveness dependence on initial value](image)

**Figure 4.9:** Robustness: influence of the initial value of the robustness index on the effectiveness of the measure. In this case, the considered measure is reduction or increase of the conditional probability of system failure.

The quantitative evaluation provides insight on the behaviour of the system's robustness when the principal design parameters are adjusted. However the values that have been found are only applicable to the specific situation that is considered in 4.3.4. Therefore an additional qualitative evaluation is provided as well.

**Increase \( C(D) \)**

Even though the qualitative analysis shows that increasing the direct consequences might be effective, this strategy is counter intuitive. It is very likely to be rejected on the basis of risk based approaches that aim to reduce risks.

**Increase \( P(D) \)**

Similar to the previous strategy, this strategy is likely to be rejected on the basis of risk based approaches. Moreover, the qualitative analysis has already shown that this strategy is not effective. This is caused by the fact that the parameter is in ally terms of the equation.

**Decrease \( C(F) \)**

Decreasing the consequences of system failure is a more intuitive strategy. To get a good understanding of this strategy, the definition of indirect consequences is required. Indirect consequences are related to system failure. As discussed in Section 4.3.3, failure refers to loss of function(s) of the system. Hence, reducing indirect consequences comes down to limiting the functionality of the system. Therefore, this strategy is very sensitive to the definition of the system (and corresponding failure consequences).

This is explained by an example: when the system is defined as the flood defense system, the consequence of failure of a navigation lock is flooding. When the system is defined as the system of main waterways, the consequence of failure of a navigation lock is delay for navigation.

**Note:** compartmentation of the structure, doesn't really result in a reduction of the consequences of system failure. Compartmentation only reduces the occurrence probability of these consequences.

**Decrease \( P(F|D) \)**

Reducing the system's failure probability seems to be the most promising strategy to increase robustness. This strategy is similar to a risk based approach in which the system is designed such that the maximum allowable risk is not exceeded. Consequences are generally fixed in such an approach. Options to execute this strategy in the design have been mentioned in Section 4.2.3. In addition to these strategies, also compartmentation of the structure can be considered as mentioned before.

Similar to the strategy of decreasing indirect consequences, this strategy depends on the definition of the
Robustness relates consequences of system failure to the initial event that caused this failure. A robust system has the ability to withstand events like fire, explosions, impact or consequences of human error, without being damaged to an extent that is disproportionate to the original cause. This disproportionality can be quantified using Formula 4.1:

\[ I_{rob} = \frac{R_{dir}}{R_{dir} + R_{ind}} \]

Section 4.4 shows that robustness can best be increased by strategies that are associated with decreasing the indirect risks. The Eurocodes (see Section 4.2.3), propose three strategies to increase the robustness that focus on decreasing the indirect risk:

- **Strategy 1** Determine on which elements the structure relies for its overall stability and design these elements as critical elements to increase the probability of survival.
- **Strategy 2** Use materials with high ductility for the design of structural elements to create a high absorption capacity of deformation energy without causing cracks.
- **Strategy 3** Provide sufficient redundancy to ensure that loads can be absorbed in alternative ways after occurrence of an accidental load.

The first strategy, is in fact the strategy that initiated the research. It aims to increase the robustness by reducing the probability of failure of the individual structural elements \( P(D \mid H) \). Based on the simplified representation in Figure 4.8, the impression is given that this strategy will not be effective. However, Figure 4.8 ignores the fact that also the structural elements, remaining after damage, are effected when \( \beta_{\text{design}} \) is increased. Hence, the strategy seems reasonable however it's exact effect has to be examined.

Subsequently, the 2nd strategy is not discussed. As will be explained in Section 5.3.3, the damage \( 'D' \) is modelled as a sudden loss of the structural element. This means that the actual behaviour of the material is neglected.

Finally, the third strategy is a well known strategy within all kinds of engineering disciplines. It reduces the failure probability of the system by duplication of critical elements: when a critical element fails, a duplicate is available to take over its functions. For successful implementation of this strategy it is important that the duplicate is able to function independently to the original element. This strategy is often expensive but very helpful in managing large risks. The selected case study contains redundant structural elements (see Chapter 5). In the analysis of the results, a rough analysis on the effect of these redundant elements is included.
Case study
An answer to the research question will be given based on a case study of lock complex ‘Sluis Weurt’. The robustness index of this structure will be calculated for multiple values of the robustness index. The robustness calculation requires input that can be found through an analysis of the system. The results from this chapter will be used in Chapter 6 and Chapter 7 to calculate the probability of damage and the probability of system failure. Subsequently, Chapter 8 uses information from this chapter to calculate the robustness indices. The output of this chapter consists of descriptions and values of:

1.) \( P(H) \) Section 5.2
2.) \( D \) and \( C(D) \) Section 5.3
3.) \( F \) and \( C(F) \) Section 5.4

The lock complex was part of the project ‘Risico Inventarisatie Natte Kunstwerken (RINK)’. A number of reports from this project has been made available by Witteveen+Bos. Amongst others, these include a Reliability Availability Maintainability and Safety (RAMS) analysis and supporting appendices [12]. In addition, the lock complex was periodically assessed in 2010 in the context of water safety regulations [26]. These reports are used as main sources for the systems analysis.

5.1. General description

First, a general description of the lock complex is given. Figure 5.1 shows a 3D impression of both the western (left) and the eastern (right) navigation lock [10]. It can be clearly seen that they are designed with different lock gates: lifting gates in the western lock and rolling gates in the eastern lock. This research focuses on the eastern navigation lock that is nearing its end of lifetime.
The lock chamber has a total length of 266.00 meter and a width of 16.00 meter. There are 3 lock heads that are constructed with identical rolling gates. Maximum retaining height at the river Waal side is NAP + 15.25 meter, this height is obtained by installing an extension piece on top of the gate.

5.1.1. Lockage procedure

The lock complex uses a special lockage procedure for high water levels. In normal conditions the third lock gate is not operational, it is only used in case water levels on the river Waal rise above NAP + 10.00 meter. From that point on a stepped lockage procedure is active to minimise the water level difference over a single lock gate. In addition, it reduces the probability of flooding due to a ship collision.

For water levels on the river Waal above NAP + 12.80 meter navigation is shut down. The navigation lock than uses the three gates for stepped retaining. This requires an additional extension piece on top of the first gate on the river Waal side. When water levels rise above NAP + 15.25 meter, the retaining function fails due to overflow and/or overtopping.

For water levels on the river Waal below NAP + 7.20 meter navigation is shut down as well. This is due to the sill of the eastern lock that is located at NAP + 3.00 meter. At that point the minimum required draught cannot be guaranteed anymore. The lockage procedures are summarised in Table 5.1 and Figure 5.3. In addition, Figure 5.4 shows how the water level difference of a single lock gate is influenced by the lockage procedure.
5.2. Hazards

Two hazards are considered: water level differences and ship collision. In Chapter 6 the damage probability per hazard is discussed. This section discusses the probability density functions of the hazards only.

5.2.1. Water level difference

The left hand side of Figure 5.4, shows the probability density function of the water levels on the river. The right hand side of Figure 5.4, shows the relation between the water level on the river Waal and the water level difference over a single lock gate. It can be clearly seen that the relation also depends on the active lockage procedure.

Implicitly, the graphs from Figure 5.4 show that only 'positive' water level differences are considered within this research. These are water level differences that are caused by high water levels on the river Waal. This means that scenarios in which failure occurs due to low water levels on the river are not considered.
The distributions from Figure 5.4 can be combined to obtain the probability density function of the water level difference \( \Delta h \). Because of the different lockage procedures at Sluis Weurt, a discontinuous function results. This distribution is shown in Figure 5.5. The derivation of this function can be found in Appendix C.

![PDF of the water level difference](image)

Figure 5.5: Hazards: probability density function of the water level difference over a single lock gate. Only water level differences caused by high water on the river Waal are considered. The probability that such a water level difference occurs is equal to 3.49E-01.

Figure 5.5 shows only a part of the probability density function: the part of the Gumbel distribution that corresponds to water levels \( h > 7.50 \text{ m} + \text{NAP} \). Lower water levels are neglected.

The water level difference creates a pressure difference on the structural element (lock gate). This pressure difference causes forces ([kNm], [kN]) in the structural elements of the lock gate. For calculation of the failure probability of the lock gate the resistance has to be expressed in the same units as the force.

### 5.2.2. Ship collision

Ship collisions do not occur frequently. But when they do occur, large consequences may result. An example is the ship collision with the weir at Grave (see Chapter 1). Based on the log of the locking complex \([12]\) the probability of a ship collision is equal to:

\[
P(\text{SC}) = 4.36 \times 10^{-6} \text{ [yr}^{-1}\text{]}\]

An important boundary condition for a ship collision at Sluis Weurt is that the water level on the river Waal has to be at least 7.50 m + NAP. When the water level drops below this threshold value, navigation is stopped due to insufficient draught. In addition, navigation is stopped for water levels on the river Waal above 12.80 m + NAP. For higher water levels, the risk of flooding is considered to be too large. At that point, only the gate at the river side has a retaining function when the extension piece is installed (see Figure 5.3).

The impact of the ship collision depends on the type of ship (mass, elasticity) and its sailing velocity. These parameters determine the energy that is involved in the collision. This research considers a single ship type: a fully loaded ‘Groot Rijnschip’ (CEMT Va). These are the largest ships that are able to pass the eastern navigation lock. The larger CEMT Vb ships pass through the western lock.

Based on the report “Aanvaarrisisco’s voor sluisdeuren” \([37]\), the collision energy on the lock gates can be approximated. The distribution of the collision energy is presented in Figure 5.6.

The right hand side of Figure 5.6 shows the approximation of the collision energy. The collision energy can be used to calculate the force that is exerted on the structure caused by the collision. This is done by modeling the ship collision as a concentrated load. Appendix C can be consulted for a more detailed explanation. The resulting collision force is presented in Figure 5.7.
5.2. Hazards

Figure 5.6: Probability distribution of the collision energy. Left: energy distribution based on ship class CEMT Va (‘Groot Rijnschip’), water depth 3.85m and varying velocity [37]. Right: approximation using a normal distribution.

In the assumed distribution, only the sailing velocity is considered as a stochastic variable. A better representation of reality could be obtained by including the mass of the ship as a stochastic variable as well. This would probably result in a probability density function with a lower mean value and a larger variance. Hence, the current assumption overestimates the load.

In practice, many other types of ships will be present. Ranging from recreational boats to various types of inland vessels. All these types of ships are included in the occurrence probability at the beginning of this section which is based on the log of navigation complex. According to this log, none of the registered collision events has led to loss of a lock gate.

Figure 5.7: Probability distribution of the ship collision force
5.3. Damage states

Many damage states are possible for "Sluis Weurt". Damage varies from small damage (e.g. broken lights) to large damage (e.g. structural failure of the chamber wall). This research focuses on one specific damage state, in which one structural component of the system is damaged. Selection of this structural component and severity of the damage are discussed in this section.

In Section 5.3.1, a physical-functional decomposition is used to identify the structural components that contribute to the main functions of the lock complex. Subsequently, in Section 5.3.2, a Failure Mode Effects and Criticality Analysis (FMECA) is performed to find the structural element that is most critical. Finally, Section 5.3.3 will explain the extent to which the critical structural element is damaged.

5.3.1. Physical decomposition

The physical decomposition of the eastern lock is extracted from the RAMS analysis [12]. Results are summarised in Table 5.2, components are already linked to their functions. A first selection of critical elements can be made based on the functions that they contribute to. In addition, interest goes out to elements that are part of the structural system and whose design is influenced by the reliability index.

Table 5.2: Damage states: physical and functional decomposition [12].

<table>
<thead>
<tr>
<th>Element / subsystem</th>
<th>Water retaining</th>
<th>Navigation</th>
<th>Structural element</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Driving system culvert slides</td>
<td>x</td>
<td>x</td>
<td>-</td>
</tr>
<tr>
<td>2 Driving system rolling gates</td>
<td>x</td>
<td>x</td>
<td>-</td>
</tr>
<tr>
<td>3 Lightning protection system</td>
<td>x</td>
<td>x</td>
<td>-</td>
</tr>
<tr>
<td>4 Operating and control system</td>
<td>x</td>
<td>x</td>
<td>-</td>
</tr>
<tr>
<td>5 Operator building</td>
<td></td>
<td>x</td>
<td>-</td>
</tr>
<tr>
<td>6 Bed protection</td>
<td>x</td>
<td>x</td>
<td>-</td>
</tr>
<tr>
<td>7 Exterior lights</td>
<td></td>
<td></td>
<td>-</td>
</tr>
<tr>
<td>8 CCTV system</td>
<td></td>
<td>x</td>
<td>-</td>
</tr>
<tr>
<td>9 Communication system</td>
<td></td>
<td>x</td>
<td>-</td>
</tr>
<tr>
<td>10 Operator building facilities</td>
<td>x</td>
<td>x</td>
<td>-</td>
</tr>
<tr>
<td>11 Soil retaining structures</td>
<td></td>
<td>x</td>
<td>yes</td>
</tr>
<tr>
<td>12 Locking chamber</td>
<td>x</td>
<td>x</td>
<td>yes</td>
</tr>
<tr>
<td>13 Low voltage installations</td>
<td>x</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td>14 Level measuring system</td>
<td>x</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td>15 Culvert pumps</td>
<td>x</td>
<td>x</td>
<td>-</td>
</tr>
<tr>
<td>16 Emergency power supply</td>
<td></td>
<td>x</td>
<td>-</td>
</tr>
<tr>
<td>17 Maintenance facility</td>
<td></td>
<td></td>
<td>-</td>
</tr>
<tr>
<td>18 Radar system</td>
<td></td>
<td>x</td>
<td>-</td>
</tr>
<tr>
<td>19 Fenders / guiding structures</td>
<td>x</td>
<td>x</td>
<td>yes</td>
</tr>
<tr>
<td>20 Shipping signals system</td>
<td>x</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td>21 Culvert gates</td>
<td>x</td>
<td>x</td>
<td>yes</td>
</tr>
<tr>
<td>22 Lock gates</td>
<td>x</td>
<td>x</td>
<td>yes</td>
</tr>
<tr>
<td>23 Lock head (concrete)</td>
<td>x</td>
<td>x</td>
<td>yes</td>
</tr>
<tr>
<td>24 Locking complex terrain</td>
<td></td>
<td></td>
<td>-</td>
</tr>
<tr>
<td>25 Shallow foundation</td>
<td>x</td>
<td>x</td>
<td>yes</td>
</tr>
<tr>
<td>26 Sheet piles</td>
<td>x</td>
<td>x</td>
<td>yes</td>
</tr>
<tr>
<td>27 Central control system</td>
<td>x</td>
<td>x</td>
<td>-</td>
</tr>
<tr>
<td>28 Bascule bridges</td>
<td>x</td>
<td></td>
<td>yes</td>
</tr>
</tbody>
</table>

From Table 5.2 the components 12, 21, 22, 23, 25 and 26 are selected for further analysis. Structural elements that are part of the holding basin (11, 19) do not have a role in flood protection and are not considered as critical elements. For example structural failure of fenders and guiding structures does result in delay for navigation however these delays are manageable and they do not result in a complete shutdown of the navigation lock. It is expected that as a result these elements will be filtered out anyway in the FMECA.
5.3.2. Failure Mode Effects and Criticality Analysis (FMECA)

The FMECA is a purely qualitative analysis that is used to determine for which elements additional quantification is required. First a description is provided for the components that have been selected from Table 5.2. This holds a description of how the component contributes to the primary functions and possible failure modes. Subsequently probability of these failure modes is estimated and the possible effects are described. Rating of probability and effects is done on a scale from 1 to 5. In which 1 is low probability/effect and 5 is high probability/effect. Two criticality scores are calculated one by summation of the ratings on probability and effect and one by multiplication of these ratings.

A detailed elaboration of the FMECA is given in Appendix D. Here, only the considered elements and results are presented.

**Locking chamber (12)**

![Figure 5.8: FMECA: Locking chamber](image)

**Culvert gates (21)**

![Figure 5.9: FMECA: Culvert gates](image)

**Lock gates (22)**

![Figure 5.10: FMECA: Lock gates](image)
Lock heads (23)

Figure 5.11: FMECA: Lock heads

Shallow foundation (25)

Figure 5.12: FMECA: Shallow foundation

Sheet piles (26)

Figure 5.13: FMECA: Sheet piles. Orange lines indicate sheet piles for piping. Light blue lines indicate sheet piles for stability.

FMECA results

Results of the failure mode effects and criticality analysis are summarised in Table 5.3. Following from the table it is concluded that the lock gates are the most critical elements of the lock complex. This is explained by the fact that they have a relatively large contribution to both functions of the navigation lock which results in large consequences of failure. Also the failure probability is large because the gates are exposed to a wide range of possible hazards and include moving elements that are susceptible to failure.

<table>
<thead>
<tr>
<th>#</th>
<th>Component</th>
<th>Failure probability</th>
<th>Failure effects</th>
<th>Criticality score</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Σ</td>
<td>Π</td>
<td>Σ</td>
</tr>
<tr>
<td>1</td>
<td>Lock gates</td>
<td>3</td>
<td>4</td>
<td>7</td>
</tr>
<tr>
<td>2</td>
<td>Shallow foundation</td>
<td>1</td>
<td>5</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>Culvert gates</td>
<td>2</td>
<td>3</td>
<td>5</td>
</tr>
<tr>
<td>4</td>
<td>Locking chamber</td>
<td>1</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Lock heads</td>
<td>1</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Sheet piles</td>
<td>1</td>
<td>4</td>
<td>5</td>
</tr>
</tbody>
</table>
5.4. System failure

From the FMECA it is concluded that the lock gates are the most critical structural elements of the lock complex. Considering the navigation function it is of no importance which of the lock gate fails; the consequences are the same for each lock gate. The flood defense function on the other hand is dominated by the lock gate in the upper head: the Waal gate. For water levels above NAP + 12.80 meter this lock gate is the only gate with a water retaining function. Hence, the Waal gate is considered as the most critical structural element. For this reason the research focuses on robustness indices that correspond to an initial damage state in which the Waal gate is lost.

5.3.3. Damage severity

Knowledge about the damage severity is important for two reasons. First of all, it determines the value of the direct consequences $C(D)$. A small scratch doesn't necessarily require direct repair. On the other hand, collapse of a supporting beam does require direct repair measures. In addition, repair of a supporting beam will cost a lot more to repair in comparison to the small scratch.

Secondly, the severity of damage influences the probability of subsequent failure of the system $P(F|D)$. A small scratch on the first gate doesn't really increase the probability of failure of the second gate. On the other hand, collapse of a supporting beam on the first gate due to water level differences does increase the probability that the second gate fails. After all, after collapse of the first supporting beam the water flows in and causes the exact same load on the 2nd gate.

Based on the decomposition and the FMECA it is determined that the lock gates are the most critical structural elements. To model the extent of damage the approach of 'sudden column loss' is used. This approach originates from building engineering in which it indicates that a hazard causes complete loss of (part of) the structural elements. In the case of "Sluis Weurt" the approach will be used as 'sudden gate loss'. This is illustrated in Figure 5.14.

The direct consequence (cost) of loss of a gate is that a new lock gate is required. Costs of this consequence are based on an interview with Mr. de Koster (engineering consultant, Rijkswaterstaat). Design, construction and installation of the new lock gate are included in this price.

$$C(D, \text{structural failure}) = \text{€10,000,000.00}$$

5.4. System failure

System failure refers to failure of the main parent systems: the flood defense system and the system of main waterways. The lock complex also provides for other functions such as cultural historical value and a small contribution to water management. The consequences of failure of these functions are limited in terms of economical value and are therefore assumed to be sufficiently robust.
5.4.1. Flooding

Sluis Weurt has a flood defense function with respect to the area of dike ring 41. Flooding of this area occurs when inflow of water at Sluis Weurt leads to water levels on the Meuse-Waal canal of NAP + 10.00 meter and higher. Storage capacity of the canal has been checked and it turns out that it is negligible (see Appendix B.3). Therefore failure of the water retaining function is defined as:

"Sluis Weurt fails when water levels on the Meuse-Waal canal become larger than NAP + 10.00 meter due to water flowing in from the river Waal through, around, under or over the lock complex."

Flooding scenarios of dike ring 41 have been studied during the project 'Veiligheid Nederland in Kaart' (VNK2, [36]). One of the scenarios is a breach at Sluis Weurt. Economic damage of flooding is estimated at €7,070,000,000 for a water level corresponding to an exceedance frequency of 1:1250 year (14.66 m + NAP). In addition, the estimated number of casualties is between 75 and 650 people. Land elevation at the lock complex is relatively high and gradually slopes down in western direction. In case of a breach, water would first flow westward towards the lowest part of the dike ring, around the towns Druten and Dreumel. From this point on, the water starts filling the dike ring and water will flow into the remaining areas as well. The flooding scenario is summarised in Figure 5.15.

Because of the different lockage procedures, flooding due to failure of Sluis Weurt might occur at different water levels. The above presented consequences correspond to a water level on the river Waal of 14.66 m + NAP. For lower water levels, the consequences of flooding will be probably not as high as described above.

A damage function is used to estimate the consequence of flooding for lower water levels. The damage function is assumed to be linear up to the point where the maximum inundation depth is reached. At that point, the damage no longer increases. See Figure 5.16.
Table 5.4: Estimated expected value of damage due to flooding

<table>
<thead>
<tr>
<th>Procedure</th>
<th>Water level</th>
<th>Expected damage of flooding</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal lockage</td>
<td>7.50 - 10.00</td>
<td>€ 0.00</td>
</tr>
<tr>
<td>Stepped lockage</td>
<td>10.00 - 12.80</td>
<td>€ 4,000,000.00</td>
</tr>
<tr>
<td>Stepped retaining</td>
<td>12.80 - 15.25</td>
<td>€ 7,070,000,000.00</td>
</tr>
</tbody>
</table>

5.4.2. Navigation delay

Sluis Weurt provides a connection between the river Waal and the Meuse-Waal canal. Failure of the navigation function is defined as:

"Sluis Weurt fails when it causes delay for navigation due to natural boundary conditions, failure of the locking process or structural failure."

The consequences of failure of a navigation delay largely depend on the duration of the delay. This duration is largely dependent on the nature of failure. Structural failure generally requires significant repairs that take up a lot of time. The mean time to repair (MTTR) has been mentioned in Section 5.3.2.

Short delay

Sluis Weurt has the benefit that it is constructed with 3 lock gates. In case one lock gate fails, there are still 2 lock gates left that can be used for lockage. Still, this causes a delay for navigation because protocol requires that there always is a gate located in the Waal head. In case the Waal gate fails, the middle gate has to be removed and floated into the position of the Waal gate. Rijkswaterstaat estimates that this procedure takes up several days, a week at most. For the remainder of this report this is considered as a 'short delay'.

Long delay

In case more than one lock gate fails, navigation can no longer make use of the lock. Both lock gates that have failed have to be repaired. In a worst case scenario it might even be possible that completely new lock gates have to be constructed. Luckily, the duration of delay of a single ship will still be limited due to the presence of the western lock gate. Still, the western lock is not equipped to handle double its capacity. Until the eastern lock is restored, ships will experience delay at the lock complex. According to section 5.3.2 the MTTR is several months. This scenario of 2 failing lock gates is referred to as 'long delay'.

Consequences

Rijkswaterstaat [38] has calculated that the mean value of time for freight transport on inland waterways is €382.00 per ship per hour. On average 3 ships pass the eastern lock every hour [12]. Due to the presence of the western navigation lock the delay per ship is assumed to be limited to 1 hour.

The expected damage of a short delay is now approximated as the number of ships that passes the lock in a period of 5 days multiplied by the mean value of time for inland navigation:

\[ C(\text{Short delay}) = \left[ \text{No. of ships in 5 days} \right] \cdot \left[ 1 \text{ hours} \cdot \€382 \right] \]

\[ C(\text{Short delay}) = [3 \cdot 24 \cdot 5] \cdot [1 \cdot 382] \]

\[ C(\text{Short delay}) = \€137,520.00 \]

The expected damage of a long delay is now approximated as the number of ships that passes the lock in a period of 4.5 months multiplied by the mean value of time for inland navigation:

\[ C(\text{Long delay}) = \left[ \text{No. of ships in 4.5 months} \right] \cdot \left[ 1 \text{ hours} \cdot \€382 \right] \]
The approach above uses a linear damage function. The damage function is presented in Figure 5.17. Also the calculated values of delay consequences are highlighted.

![Damage function, navigation](image)

Figure 5.17: Damage function of navigation delay. Boxes indicate the corresponding lockage procedure. Black dots represent the assumed expected value of the damage (see also Table 5.4). Shape of the damage function is based on [31].

Table 5.5: Estimated expected value of damage due to navigation delay

<table>
<thead>
<tr>
<th>Type</th>
<th>Duration</th>
<th>Estimated damage of delay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Short delay</td>
<td>~ 5 days</td>
<td>€ 137,520.00</td>
</tr>
<tr>
<td>Long delay</td>
<td>~ 4.5 months</td>
<td>€ 3,713,040.00</td>
</tr>
</tbody>
</table>
6

Damage probabilities

This chapter provides the framework that is used for calculation of damage probabilities. Section 6.1 introduces the main aspects of calculating the damage probability for multiple hazards. Subsequently, in Section 6.2 the resistance model of the structural design of the lock gates is presented. This model is used in Section 6.3 to perform an example calculation for one specific value of the reliability requirement ($\beta_{\text{design}}$). Finally, in Section 6.4, the damage probabilities are presented for a larger range of values of the reliability requirement.

6.1. Damage probability for multiple hazards

In general, structures are subjected to multiple hazards. Each hazard has its own probability density function (PDF) which may have any form. An example of multiple hazards is illustrated in Figure 6.1. Also the PDF of the resistance (R) of a structural element is illustrated. The probability that the structural element fails is equal to the probability that the resistance is smaller than the load. This probability differs per hazard.

In order to satisfy the reliability requirement $\beta_{\text{design}}$ for all loads, the governing load is identified. Subsequently, the resistance (R) of the structural element is designed such that the requirement is met for this governing load. Hence, the reliability requirement gives a direct relation between governing load and resistance. Through the resistance, the reliability requirement is indirectly related to other loads.

Figure 6.1: Example illustration of multiple hazards (H1 and H2) and resistance of a structural element (R) with different probability density functions. Left: initial situation. Right: the effect of increasing the reliability requirement $\beta_{\text{design}}$. 
Remark Figure 6.1:
For designers it seems counter intuitive that the resistance shifts instead of the load. In design procedures, a partial factor on the load is used to achieve a larger reliability. However, reality is that the load itself doesn't change. Only the design value of the load is increased. The increased design value of the load results in a stronger structure. Hence, in practice the resistance of a structural element is increased to achieve a larger reliability.

In Figure 6.2, the green boxes (1) indicate the fixed input parameters. Yellow boxes (2) indicate the parameters that have to be calculated. The blue box (3) of the reliability requirement is a varying input parameter. To calculate the damage probability it is important that both load and resistance are expressed in the same units. This can be achieved by using a model of the structural design to calculate normal forces [kN], shear forces [kN] and bending moments [kNm] that result from each hazard.

In summary, in order to calculate the damage probability, it is important to have good knowledge of the probability density functions of all hazards and the resistance. In addition a model of the structural design is required.

Figure 6.2: Damage probabilities: calculation process to determine the failure probability per hazard.
6.2. Simplified model of the resistance

In the previous section it has been concluded that, in order to calculate the damage probability, it is important to have good understanding of the structural design. This section explains the calculation on the basis of the lock complex "Sluis Weurt". First, the structural design and its resistance are discussed. Subsequently, an overview of the considered hazards and their probability density functions is given. Finally, the damage probabilities are calculated for each hazard.

According to the FMECA (see Section 5.3.2), the lock gates are the critical structural elements. The lock gates can be modeled as a steel plate that is supported by horizontal beams, the beams transfer the loads to the supports that transfer the loads to the lock heads. See also Figure 6.3.

![Figure 6.3: Damage probabilities: structural model of the lock gates of Sluis Weurt.](image)

The material of which the lock gates are constructed is steel S355 [26]. The horizontal beams have a span of 16.70 meters and are supported at their ends. All dimensions are presented in Figure 6.4.

![Figure 6.4: Damage probabilities: dimensions of the lock gates of Sluis Weurt.](image)

Within this research, the bending moment resistance of the horizontal beams will be considered. The bending moment capacity of the beams is equal to:

\[
M_{Rd} = f_y \cdot W_{y,el}
\]

where

- \( M_{Rd} \) = Bending moment capacity [kN\text{m}]
- \( f_y \) = Ultimate tensile strength [MPa]
- \( W_{y,el} \) = Elastic section modulus [m^3]

The resistance \( M_{Rd} \) does not have a fixed value. Instead, it is designed such that it satisfies the reliability requirement with respect to the governing load (Figure 6.2). Similar to the loads, the resistance has a probability
density function as well. Its mean value ($\mu_R$) and standard deviation ($\sigma_R$) are determined by the distributions of the parameters in Formula 6.1. From literature [20] it is known that the coefficient of variation $V_X$ of the resistance will be constant. This coefficient of variation can be defined as the ratio of the standard deviation and the mean value, see Figure 6.2.

\[ V_X = \frac{\sigma_X}{\mu_X} \]

Assuming that the tensile strength and section modulus are normally distributed variables, the coefficient of variation of the bending moment capacity $M_{Rd}$ can be calculated using Formula 6.3:

\[ V^2(M_{Rd}) = V^2(f_u) + V^2(W_{y,el}) \]

The Probabilistic Model Code of the JCSS [15] provides estimated values of the coefficient of variation for both the tensile strength and the section modulus. A log-normal distribution is recommended. Within this research the same values of the coefficient of variation are assumed for a normal distribution. See Table 6.1.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Distribution type</th>
<th>Coeff. of Variance $V_X$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_u$</td>
<td>Normal</td>
<td>0.04</td>
</tr>
<tr>
<td>$W_{y,el}$</td>
<td>Normal</td>
<td>0.04</td>
</tr>
<tr>
<td>$M_{Rd}$</td>
<td>Normal</td>
<td>0.057</td>
</tr>
</tbody>
</table>

Based on Table 6.1 and Formula 6.3 we can now calculate the coefficient of variation of the resistance:

\[ V^2(M_{Rd}) = 0.04^2 + 0.04^2 \]

\[ V(M_{Rd}) = 0.057 \]

In the remainder of this report the resistance for each reliability requirement will be calculated using this value of the coefficient of variation. The resistance can be said to be normally distributed with the following parameters:

\[ F_R(M_{Rd}) = \frac{1}{2} \left[ 1 + \text{erf} \left( \frac{M_{Rd} - \mu}{\sigma \sqrt{2}} \right) \right] \]

\[ \mu = \mu(M_{Rd}) \]

\[ \sigma \approx 0.057 \cdot \mu(M_{Rd}) \]

As a result, the standard deviation increases slightly when the mean value increases. And vice versa, it decreases slightly when the mean value decreases. This is illustrated in Figure 6.5.

![PDF - Resistance for multiple values of $\mu_R$](image)

Figure 6.5: Example plots of the resistance's probability density function. An increasing mean value, results in a larger standard deviation. Hence, the top of the distribution drops and the base of the distribution widens.
6.3. Damage probability, example calculation for $\beta_{\text{design}} = 3.00$

For the reliability requirement $\beta_{\text{design}} = 3.0$, the water level difference is assumed as the governing load. At the end of this section, it is verified whether this assumption is justified or not. Following the procedure from Figure 6.2, the required resistance of the supporting beam of the lock gate is derived for water level differences.

When only a single hazard is considered, the probability of failure and the reliability requirement are directly related through the standard normal distribution (see also Chapter 3). In this case, the relation between the probability of failure due to water level differences and the reliability requirement is given by Formula 6.6:

$$\Phi(-\beta_{\text{design}}) = P(D \cap H)$$

Subsequently, the probability density of the resistance is used to determine the failure probability due to ship collision. By comparing the failure probability for both hazards, the governing load can be identified.

6.3.1. Water level differences

The probability density function of water level differences has been discussed in Section 5.2. These water level differences cause a bending moment in the supporting beams of the lock gates. The critical beam was already indicated in Figure 6.4b. The bending moments that are caused by water level differences can be calculated using Formula 6.7.

$$M_{\text{Ed}} = \frac{1}{8} \cdot \Delta h \cdot x_{\text{c.t.c.}} \cdot \rho_w \cdot g \cdot l^2$$

- $\Delta h$ = Water level difference [m] (Deterministic 3.00 - 5.00)
- $x_{\text{c.t.c.}}$ = Center to center distance beams [m] (Deterministic 2.60 - 3.60)
- $\rho_w$ = Density of the water [kg⋅m$^{-3}$] (Deterministic 1000 - 1010)
- $g$ = Gravity [m⋅s$^{-2}$] (Deterministic 9.81 - 9.82)
- $l$ = Length of the supporting beam [m] (Deterministic 16.70 - 17.00)

By combining Formula 6.7 and the probability density function of the water level differences (Section 5.2), the probability density of the bending moments is obtained. The result is shown in Figure 6.6. Derivation of this distribution can be found in Appendix C.

![Figure 6.6: (Partial) PDF of the effective bending moment caused by water level difference](image)
Now, the probability distributions of both the resistance and the load are known. The probability distribution of the resistance is given by Formula 6.5. Both distributions are used to calculate the probability of failure of the supporting beam. This failure probability can be written as:

\[
P(D \cap WD) = \int_{-\infty}^{\infty} F_R(M) \cdot f_{WD}(M) \, dM
\]

In the previous section (6.2), an assumption for the coefficient of variation of the resistance has been made. Hence, there is only one unknown parameter in Formula 6.8. This is the mean value of resistance \( \mu_R \).

The reliability requirement, \( \beta_{\text{design}} \), is equal to 3.00. Its corresponding maximum allowable probability of failure, is equal to:

\[
\Phi(-\beta_{\text{design}}) = \Phi(-3.0) = 1.35 \times 10^{-3} \ \text{[yr}^{-1}] \]

To satisfy the requirement, the maximum allowable probability of failure should not be exceeded. Hence, the mean value of the resistance \( \mu_R \) should be chosen such that the following equation should be satisfied:

\[
\int_{-\infty}^{\infty} F_R(M) \cdot f_{WD}(M) \, dM \leq 1.35 \times 10^{-3} \ \text{[yr}^{-1}] \]

Formula 6.10 is solved analytically, using Maple. It can be shown, that the requirement is satisfied when the following values are used for the probability density function of the resistance:

\[
\begin{align*}
\mu_{R,3.00} &= 2330.1 \ \text{[kNm]} \\
\sigma_{R,3.00} &= 132.8 \ \text{[kNm]}
\end{align*}
\]

Figure 6.7 gives an illustration of both functions that are in Formula 6.8. Note that two vertical axes are used with different scale. The resistance that has been found, is sufficient to satisfy the reliability requirement with respect to water level differences. In the following section, it will be checked whether this is also true for the reliability with respect to ship collision.

Figure 6.7: The shaded area indicates the part of the PDF of the load that overlaps with the fragility curve of the resistance. The probability of failure is obtained by applying Formula 6.8.
6.3.2. Ship collision

The probability density function of ship collisions has been discussed in Section 5.2. A ship collision causes a bending moment in the supporting beams of the lock gates. It is assumed that the force from a ship collision is concentrated at mid-span of one of the supporting beams. This results in the maximum bending moment and can be calculated using Formula 6.12.

The assumption that the force is concentrated on only one supporting beam is conservative. In reality, multiple beams will be addressed due to shape of the hull and (plastic) load redistribution.

\[
M = \frac{1}{4} F_{\text{ship}} \cdot l
\]

\[
F_{\text{ship}} = \text{Ship collision force [kN]}
\]

\[
l = \text{Length of the beam [m]}
\]

By combining Formula 6.12 and the probability density function of ship collisions (Section 5.2), the probability density of the bending moments is obtained. The result is shown in Figure 6.8. Derivation of this distribution can be found in Appendix C.

![Figure 6.8: PDF of the bending moment caused by ship collision](image)

The resistance of the supporting beams is known from the previous section. It is normally distributed with mean value and standard deviation equal to:

\[
\mu_{R,3.00} = 2330.1 \text{ [kNm]}
\]

\[
\sigma_{R,3.00} = 132.8 \text{ [kNm]}
\]

The probability of failure, due to a ship collision, can be calculated by using Formula 6.14.

\[
P(D \cap SC) = P(SC) \cdot \int_{-\infty}^{\infty} F_R(M) \cdot f_{SC}(M) \, dM
\]

Note that, in contrast to the failure probability due to water level differences, the occurrence probability is included. In case of water level differences, the probability density is given for the occurrence of water level differences. These water level differences can be directly related to the occurring bending moment. For ship collision, the occurrence probability is not yet included in the probability density of the bending moment.

From Section 5.2, we know that:

\[
P(SC) = 4.36E-06
\]
Maple is used to evaluate the integral from Formula 6.14 analytically. The result is that:

\[
(6.15) \quad P(D \cap SC) = 4.36 \times 10^{-06} \cdot \int_{-\infty}^{\infty} F_{R}(M) \cdot f_{SC}(M) \, dM \approx 4.36 \times 10^{-06}
\]

Figure 6.9 gives an illustration of both functions that are in Formula 6.14. Note that two vertical axes are used with different scale. The resistance that has been found, is insufficient to resist loads that are caused by ship collision. However, due to the low probability of occurrence, the reliability with respect to ship collisions is still sufficient.

\[
\text{Figure 6.9: The shaded area indicates where the PDF of the load overlaps with the fragility curve of the resistance. The probability of failure is obtained by applying Formula 6.14. The function } f_{SC}(M) \text{ describes the bending moment given the fact that a ship collision occurs. Hence, to calculate the probability of damage, the result should be multiplied by the probability of occurrence (see also Formula 6.14).}
\]

6.3.3. Governing load check

A final check is performed, to verify that the water level difference indeed is the governing load. This requires a comparison of the damage probability with respect to both loads. Water level difference is the governing load when the following relation is true:

\[
(6.16) \quad P(D \cap WD) > P(D \cap SC)
\]

\[
1.35 \times 10^{-03} > 4.36 \times 10^{-06}
\]

For the case of this example calculation, the statement is true. Hence, the water level difference indeed is the governing load on the supporting beam. The corresponding resistance is insufficient to resist loads that are caused by ship collision. However, due to the low probability of occurrence, the reliability with respect to ship collision is still sufficient.

When the reliability requirement is increased, the resistance has to be increased as well to satisfy the requirement. The failure probability is governed by the overlapping tails of the PDF of the load and the CDF of the resistance. By increasing the resistance, the overlap of the tails is decreased.

In the example above, the overlap between the CDF of the resistance and the PDF of ship collision is approximately 100%. This is allowed, because the reliability requirement is satisfied by the low probability of occurrence. However, at a certain value of the reliability requirement, the maximum allowable failure probability is lower than the probability of occurrence of a ship collision. At this point, the probability of failure due to ship collision has to be decreased by decreasing the overlap of the PDF of the load and the CDF of the resistance. This might require a significant jump in the required resistance.
6.4. Damage probabilities for other values of $\beta_{\text{design}}$

In correspondence with the procedure from Section 6.3, damage probabilities can be calculated for a wider range of values of the reliability requirement. First, the required resistance for each value of $\beta_{\text{design}}$ is presented. Subsequently, the damage probability is presented per hazard.

In Chapter 3, it has been determined that the research focuses on $\beta_{\text{design}}$-values between 2.00 and 6.00. The range contains values that are used in practice, plus additional values just outside this traditional scope. The relation between $\beta_{\text{design}}$ and the allowable failure probability is illustrated in Figure 6.10.

6.4.1. Required resistance per value of $\beta_{\text{design}}$

For each value of $\beta_{\text{design}}$ the required resistance of the lock gates is presented in Table 6.2 and Figure 6.11.

Table 6.2: Values of the required resistance to satisfy the reliability requirement $\beta_{\text{design}}$. Values are plotted in Figure 6.11. * The actual reliability is calculated by adding up the damage probability of each hazard. The small differences with the required reliability are neglected.

<table>
<thead>
<tr>
<th>$\beta_{\text{design}}$</th>
<th>$P_f$ [yr$^{-1}$]</th>
<th>$\mu_R$ [kNm]</th>
<th>$\sigma_R$ [kNm]</th>
<th>$P$ (D due WD)</th>
<th>$P$ (D due SC)</th>
<th>Actual $\beta$ [yr$^{-1}$]*</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.00</td>
<td>2.28E-02</td>
<td>1,901</td>
<td>108</td>
<td>2.28E-02</td>
<td>4.36E-06</td>
<td>2.00</td>
</tr>
<tr>
<td>2.25</td>
<td>1.22E-02</td>
<td>2,048</td>
<td>117</td>
<td>1.22E-02</td>
<td>4.36E-06</td>
<td>2.25</td>
</tr>
<tr>
<td>2.50</td>
<td>6.21E-03</td>
<td>2,159</td>
<td>123</td>
<td>6.21E-03</td>
<td>4.36E-06</td>
<td>2.50</td>
</tr>
<tr>
<td>2.75</td>
<td>2.98E-03</td>
<td>2,250</td>
<td>128</td>
<td>2.98E-03</td>
<td>4.36E-06</td>
<td>2.75</td>
</tr>
<tr>
<td>3.00</td>
<td>1.35E-03</td>
<td>2,330</td>
<td>133</td>
<td>1.35E-03</td>
<td>4.36E-06</td>
<td>3.00</td>
</tr>
<tr>
<td>3.25</td>
<td>5.77E-04</td>
<td>2,405</td>
<td>137</td>
<td>5.77E-04</td>
<td>4.36E-06</td>
<td>3.25</td>
</tr>
<tr>
<td>3.50</td>
<td>2.33E-04</td>
<td>2,476</td>
<td>141</td>
<td>2.33E-04</td>
<td>4.36E-06</td>
<td>3.50</td>
</tr>
<tr>
<td>3.75</td>
<td>8.84E-05</td>
<td>2,545</td>
<td>145</td>
<td>8.84E-05</td>
<td>4.36E-06</td>
<td>3.74</td>
</tr>
<tr>
<td>4.00</td>
<td>3.17E-05</td>
<td>2,613</td>
<td>149</td>
<td>3.17E-05</td>
<td>4.36E-06</td>
<td>3.97</td>
</tr>
<tr>
<td>4.25</td>
<td>1.07E-05</td>
<td>2,680</td>
<td>153</td>
<td>1.07E-05</td>
<td>4.36E-06</td>
<td>4.17</td>
</tr>
<tr>
<td>4.50</td>
<td>3.40E-06</td>
<td>25,193</td>
<td>1,436</td>
<td>0.00E+00</td>
<td>3.40E-06</td>
<td>4.50</td>
</tr>
<tr>
<td>4.75</td>
<td>1.02E-06</td>
<td>30,282</td>
<td>1,726</td>
<td>0.00E+00</td>
<td>1.02E-06</td>
<td>4.75</td>
</tr>
<tr>
<td>5.00</td>
<td>2.87E-07</td>
<td>33,119</td>
<td>1,888</td>
<td>0.00E+00</td>
<td>2.87E-07</td>
<td>5.00</td>
</tr>
<tr>
<td>5.25</td>
<td>7.60E-08</td>
<td>35,406</td>
<td>2,018</td>
<td>0.00E+00</td>
<td>7.60E-08</td>
<td>5.25</td>
</tr>
<tr>
<td>5.50</td>
<td>1.90E-08</td>
<td>37,438</td>
<td>2,134</td>
<td>0.00E+00</td>
<td>1.90E-08</td>
<td>5.50</td>
</tr>
<tr>
<td>5.75</td>
<td>4.46E-09</td>
<td>39,332</td>
<td>2,242</td>
<td>0.00E+00</td>
<td>4.46E-09</td>
<td>5.75</td>
</tr>
<tr>
<td>6.00</td>
<td>9.87E-10</td>
<td>41,143</td>
<td>2,345</td>
<td>0.00E+00</td>
<td>9.87E-10</td>
<td>6.00</td>
</tr>
</tbody>
</table>

The general conclusion from the data, is that a larger resistance is required for higher reliability requirements. The required increase of resistance happens with gradual steps. However, in the interval where the reliability requirement $\beta_{\text{design}}$ is increased from 4.25 to 4.50, a significant increase in resistance is required.
This jump is caused by the fact that there is a shift in the governing load. For values of $\beta_{\text{design}} \leq 4.25$, water level differences are the governing load. As long as this is the case, relatively small increments of the resistance are required to reduce the overlap between the PDF of the load and the CDF of the resistance.

For values of $\beta_{\text{design}} \geq 4.50$, the water level differences no longer are the governing load. This is caused by the fact that the occurrence probability of a ship collision is larger than the maximum allowable failure probability. In other words, the reliability with respect to ship collision is insufficient:

$$
\beta = -\Phi^{-1}(4.36E-06) \approx 4.45 \leq 4.50
$$

Hence, the strength has to be increased in order to resist forces that are caused by ship collision. The required increase in resistance is almost a factor 10. This substantial increase is caused by the relatively low forces of water level differences (case specific) and the relatively large forces of ship collision (only heaviest ships are considered). When all possible (ship) masses are considered, the increase in required resistance would be more gradual. Figure 6.12 provides an illustration of the required increase of the resistance.
6.4. Damage probabilities for other values of $\beta_{\text{design}}$

6.4.2. Damage probabilities per hazard

Based on the resistances that have been calculated in Section 6.4, the damage probabilities can be calculated for each hazard. Damage refers to the damage state of the system, which is modeled as failure of the lock gate. The general formula to calculate the damage probability was given in Chapter 4, and can be written as:

\[
P(D \cap H) = P(H) \cdot P(D | H)
\]

Water level differences

As will be explained in Chapter 7, the probability of failure of the damaged system differs per lockage procedure. Therefore, the damage probabilities are calculated per lockage procedure as well. The results are given in Table 6.3 and plotted in Figure 6.13.

Table 6.3: Probability of damage due to water level differences per year. See Figure 6.13 for a graphical representation.

<table>
<thead>
<tr>
<th>$\beta_{\text{design}}$</th>
<th>$\Phi(\beta_{\text{design}})$</th>
<th>Normal lockage 7.50 - h - 10.00</th>
<th>Stepped lockage 10.00 - h - 12.80</th>
<th>Stepped retaining 12.80 - h - 15.25</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.00</td>
<td>2.28E-02</td>
<td>1.65E-02</td>
<td>3.30E-03</td>
<td>8.98E-04</td>
</tr>
<tr>
<td>2.25</td>
<td>1.22E-02</td>
<td>8.62E-03</td>
<td>3.09E-03</td>
<td>4.52E-04</td>
</tr>
<tr>
<td>2.50</td>
<td>6.21E-03</td>
<td>4.10E-03</td>
<td>1.82E-03</td>
<td>2.31E-04</td>
</tr>
<tr>
<td>2.75</td>
<td>2.98E-03</td>
<td>1.79E-03</td>
<td>1.04E-03</td>
<td>1.14E-04</td>
</tr>
<tr>
<td>3.00</td>
<td>1.35E-03</td>
<td>7.18E-04</td>
<td>5.51E-04</td>
<td>5.27E-05</td>
</tr>
<tr>
<td>3.25</td>
<td>5.77E-04</td>
<td>2.67E-04</td>
<td>2.71E-04</td>
<td>2.26E-05</td>
</tr>
<tr>
<td>3.50</td>
<td>2.33E-04</td>
<td>9.28E-05</td>
<td>1.22E-04</td>
<td>8.99E-06</td>
</tr>
<tr>
<td>3.75</td>
<td>8.84E-05</td>
<td>3.02E-05</td>
<td>5.11E-05</td>
<td>3.33E-06</td>
</tr>
<tr>
<td>4.00</td>
<td>3.17E-05</td>
<td>9.23E-06</td>
<td>1.97E-05</td>
<td>1.15E-06</td>
</tr>
<tr>
<td>4.25</td>
<td>1.07E-05</td>
<td>2.66E-06</td>
<td>7.05E-06</td>
<td>3.71E-07</td>
</tr>
<tr>
<td>4.50</td>
<td>3.40E-06</td>
<td>~0.00E-00</td>
<td>~0.00E-00</td>
<td>~0.00E-00</td>
</tr>
<tr>
<td>4.75</td>
<td>1.02E-06</td>
<td>~0.00E-00</td>
<td>~0.00E-00</td>
<td>~0.00E-00</td>
</tr>
<tr>
<td>5.00</td>
<td>2.87E-07</td>
<td>~0.00E-00</td>
<td>~0.00E-00</td>
<td>~0.00E-00</td>
</tr>
<tr>
<td>5.25</td>
<td>7.60E-08</td>
<td>~0.00E-00</td>
<td>~0.00E-00</td>
<td>~0.00E-00</td>
</tr>
<tr>
<td>5.50</td>
<td>1.90E-08</td>
<td>~0.00E-00</td>
<td>~0.00E-00</td>
<td>~0.00E-00</td>
</tr>
<tr>
<td>5.75</td>
<td>4.46E-09</td>
<td>~0.00E-00</td>
<td>~0.00E-00</td>
<td>~0.00E-00</td>
</tr>
<tr>
<td>6.00</td>
<td>9.87E-10</td>
<td>~0.00E-00</td>
<td>~0.00E-00</td>
<td>~0.00E-00</td>
</tr>
</tbody>
</table>

The results show that the probability of damage due to water level becomes approximately zero when values for $\beta_{\text{design}} \geq 4.50$. This is explained by the fact that at this point, ship collision becomes the governing load. This means that the resistance has to be increased significantly, as was explained in the previous section. Apparently, the magnitude of this increase is so large, that the probability of damage due to water level differences becomes negligible.

Figure 6.13: Probability of damage due to water level differences. See Table 6.3 for the used values.
6. Damage probabilities

Ship collision

The probability of damage due to a ship collision is presented in Table 6.4 for each considered value of $\beta_{\text{design}}$. The values are plotted in Figure 6.14.

Table 6.4: Probability of damage due to ship collision per year. See Figure 6.14 for a graphical representation.

| $\beta_{\text{design}}$ | $\Phi(\beta_{\text{design}})$ | $P(H)$ | $P(D|H)$ | $P(D \cap H)$ |
|--------------------------|-------------------------------|--------|----------|---------------|
| 2.00                     | 2.28E-02                      | 4.36E-06 | ~1.00E+00 | 4.36E-06 |
| 2.25                     | 1.22E-02                      | 4.36E-06 | ~1.00E+00 | 4.36E-06 |
| 2.50                     | 6.21E-03                      | 4.36E-06 | ~1.00E+00 | 4.36E-06 |
| 2.75                     | 2.98E-03                      | 4.36E-06 | ~1.00E+00 | 4.36E-06 |
| 3.00                     | 1.35E-03                      | 4.36E-06 | ~1.00E+00 | 4.36E-06 |
| 3.25                     | 5.77E-04                      | 4.36E-06 | ~1.00E+00 | 4.36E-06 |
| 3.50                     | 2.33E-04                      | 4.36E-06 | ~1.00E+00 | 4.36E-06 |
| 3.75                     | 8.84E-05                      | 4.36E-06 | ~1.00E+00 | 4.36E-06 |
| 4.00                     | 3.17E-05                      | 4.36E-06 | ~1.00E+00 | 4.36E-06 |
| 4.25                     | 1.07E-05                      | 4.36E-06 | ~1.00E+00 | 4.36E-06 |
| 4.50                     | 3.40E-06                      | 4.36E-06 | 7.79E-01  | 3.40E-06 |
| 4.75                     | 1.02E-06                      | 4.36E-06 | 2.33E-01  | 1.02E-06 |
| 5.00                     | 2.87E-07                      | 4.36E-06 | 6.57E-02  | 2.87E-07 |
| 5.25                     | 7.60E-08                      | 4.36E-06 | 1.74E-02  | 7.60E-08 |
| 5.50                     | 1.90E-08                      | 4.36E-06 | 4.36E-03  | 1.90E-08 |
| 5.75                     | 4.46E-09                      | 4.36E-06 | 1.02E-03  | 4.46E-09 |
| 6.00                     | 9.87E-10                      | 4.36E-06 | 2.26E-04  | 9.87E-10 |

As long as $\beta_{\text{design}} \leq 4.25$, the probability of damage due to ship collision is constant. In other words: even though the resistance of the Waal gate is increased, a ship collision will always cause failure. Thus, for values $\beta_{\text{design}} \leq 4.25$, the probability of failure due to ship collision solely depends on the occurrence probability.

The low occurrence probability of ship collision provides a minimum level of reliability, see Formula 6.17. At the point where a stricter reliability is required, the resistance is increased such that the failure probability due to a ship collision is reduced.

![Figure 6.14: Probability of damage due to ship collision: $P(D \cap SC) = P(SC) \cdot P(D|SC)$ - Based on Table 6.4.](image)

Comparison of both hazards

The graphs from Figure 6.13 and Figure 6.14 are plotted together in Figure 6.15. In addition, the reliability requirement is plotted against its corresponding allowable probability. It can be clearly seen that values of $\beta_{\text{design}} \leq 4.25$, correspond to water level differences. Values for $\beta_{\text{design}} \geq 4.50$, correspond to ship collision.
In order to satisfy the reliability requirement for a structural element, the resistance of the element has to be increased. When only a single hazard is considered, the required increase of the resistance is directly related to the standard normal distribution:

\[
\Phi(-\beta_{\text{design}}) = P(D \cap H)
\]

For the case study, a large jump is found in the required resistance. This jump occurs at the point where the governing load shifts from water level difference to ship collision. For values \(\beta_{\text{design}} \leq 4.25\), water level differences are governing when designing the resistance of the supporting beams. For these values of \(\beta_{\text{design}}\), the low probability of occurrence ensures that the structural elements are sufficiently reliable with respect to a ship collision.

When the required reliability increases further (\(\beta_{\text{design}} \geq 4.50\)), a jump occurs. The low probability of occurrence of a ship collision is no longer sufficient to ensure sufficient reliability. Hence, the resistance of the supporting beams has to be adjusted with respect to the load caused by ship collisions. Loads that are caused by ship collision are approximately a factor 10 larger than loads that are caused by water level differences. Therefore, the resistance has to be increased significantly which causes the jump.

Due to the required increase of the resistance for \(\beta_{\text{design}} \geq 4.50\), the probability of damage due to water level differences becomes approximately zero. This is enhanced by the fact that the probability density function for water level differences is truncated for \(h > 15.25 \text{ m} + \text{NAP}\).

The jump in the case study, is partly the result of the assumptions that have been made. The ship collision energy is based on the heaviest portion of possible vessels. In addition, the forces are based on the assumption that they are exerted on a single supporting beam. Another explanation of the jump, is the fact that the water level differences are relatively small. This is caused by the number of lock gates and the lockage procedures.

More general, it can be stated that a jump will occur in the required resistance when another hazard becomes governing. This jump depends on the relative magnitude of the considered loads. Due to made assumptions, the jump that was found in the case study is relatively large. The jump is likely to be smaller for navigation locks with larger water level differences.

\[(a)\] Other options are reducing the expected value of the load or the probability of occurrence. These options have not been included in this research.

---

**Figure 6.15:** Probability of damage due to individual hazards.
System failure probabilities

This chapter explains the calculation framework for the probability of system failure. Generally, system failure probabilities are calculated for undamaged systems. Within the framework of robustness, we are interested in the failure probability of the damaged system. As it turns out, the probability of failure of the damaged system is different for different causes of damage.

First, the concept of Fault Tree Analysis (FTA) is introduced briefly in Section 7.1. Subsequently, in Section 7.2 it is elaborated how correlations influence the reliability of a system. Also, relevant correlations for the case studies are discussed.

In Sections 7.3 and 7.4, the fault trees of respectively the flood defense system and the navigation are discussed. Finally, Section 7.5 gives the calculated results for all values of the reliability requirement.

7.1. Fault tree analysis (FTA)

Fault tree analysis is extremely helpful in analysing and illustrating the failure probability of the system. A fault tree gives an overview of the successive events that lead to an undesired top event (failure). Figure 7.1 shows a simplified example of a navigation lock with top event 'flood ing'.

---

Figure 7.1: Example of a fault tree.
Top events in the fault trees relate to system failure. As discussed in the system analysis (Chapter 5), two main functions are considered for Sluis Weurt: the flood defense function and the navigation function. Hence, two top events have to be considered. Each top event requires a separate fault tree.

Because the event probabilities in the fault tree depend on the reliability requirement, the probability of the top event is also expected to depend on the reliability requirement. Therefore, a separate calculation has to be performed for each value of the reliability requirement. Fault trees are constructed using the software package Isograph Reliability Workbench 11.0. The fault trees are only used to illustrate the (series of) events that lead to the top event. Resulting probabilities are calculated in Microsoft Excel.

### 7.2. Correlation

Correlation describes the mutual dependency of two stochastic variables. When these mutual dependencies are strong, they have a significant influence on the reliability of systems. Mutual dependency can be expressed as the Pearson’s product moment correlation coefficient $\rho$. The value of the coefficient could be somewhere between -1 and +1. The correlation coefficient is illustrated in Figure 7.2 by using Venn diagrams. The figure shows two failure events: F1 and F2.

<table>
<thead>
<tr>
<th>Case</th>
<th>Mutually exclusive</th>
<th>Independent</th>
<th>Dependent</th>
</tr>
</thead>
<tbody>
<tr>
<td>Correlation coefficient $\rho_{F1,F2}$</td>
<td>-1</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>Venn diagram</td>
<td>F1 F2</td>
<td>F1 F2</td>
<td>F1 F2</td>
</tr>
</tbody>
</table>

Figure 7.2: Correlation coefficients illustrated as Venn diagrams [20].

#### 7.2.1. Correlation and system reliability

Whether correlation has a positive or a negative effect on the system reliability, depends on the system. The reliability of parallel systems increases when its elements are negatively correlated. For series systems, exactly the opposite applies. This is illustrated in Figure 7.3.

The system failure probability can be exactly calculated for the three standard cases from Figure 7.2. For other values of the correlation coefficient the system failure probability can be estimated by using approximation methods such as the one formulated by Grigoriu & Turkstra [33]. They derived Formula 7.1 to calculate the reliability of a parallel system with correlated elements:

$$
\beta_S = \beta_E \sqrt{\frac{n}{1 + \rho \cdot (n-1)}}
$$
7.2. Correlation

\[ \beta_S = \text{Reliability index of the system} \]
\[ \beta_E = \text{Reliability index of all individual elements} \]
\[ n = \text{Number of elements in the system} \]
\[ \rho = \text{Correlation coefficient} \]

In other words, Formula 7.1 gives the joint failure probability of 'n' elements. This implies that the formula could also be used to calculate the failure probability of series systems. In the case of series system, the system failure probability is the sum of failure probabilities of the individual elements minus their joint failure probability.

Figure 7.4 gives an illustration of the approach by Grigoriu & Turkstra. It clearly shows that the failure probability of parallel systems decreases when the number of elements increases and/or the correlation coefficient decreases.

7.2.2. Correlations Sluis Weurt

Focus of the research is on the lock gates. For the water retaining function, the lock gates can be modeled as a parallel system: all three gates have to fail before flooding occurs. On the other hand, the lock gates can be modeled as a series system for the navigation function: failure of single lock gate leads to delay of navigation.

The lock gates have an identical design, therefore the probability density function of the resistance is the same. The probability functions of the loads however, are not the same for each lock gate. Estimations of the correlation coefficient are given, per hazard, in Table 7.1. The correlation coefficient is given as \( \rho_{F_i,F_j} \). The indices denote the failure events of Gate \( i \) (\( F_i \)) and Gate \( j \) (\( F_j \)).
Table 7.1: Hazards: estimated values for the correlation coefficient $\rho$ between failure of Gate $i$ and Gate $j$

<table>
<thead>
<tr>
<th>$\rho(F_i, F_j)$</th>
<th>$P(\text{SC } \cap \text{Gate } j)$</th>
<th>$P(\text{WD } \cap \text{Gate } j)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P(\text{SC } \cap \text{Gate } i)$</td>
<td>-0.5</td>
<td>0.0</td>
</tr>
<tr>
<td>$P(\text{WD } \cap \text{Gate } i)$</td>
<td>0.0</td>
<td>0.9</td>
</tr>
</tbody>
</table>

**Ship collision**

When a ship collision leads to failure of one of the lock gates, the ship decelerates and its kinetic energy (collision force) decreases. Thus, it is unlikely that the ship causes failure of one of the remaining lock gates. In addition, the ship blocks the waterway and navigation is delayed. This strongly reduces the failure probability due to a ship collision of the other lock gates. Based on these considerations, the correlation coefficient is assumed to be:

$$\rho(F_{i,SC}, F_{j,SC}) = -0.5$$

The exact value of the correlation coefficient is hard to approximate. Nonetheless, based on Figure 7.3, it can be concluded that the effect of values close to assumed value will have the same effect.

**Water level difference**

Once a lock gate fails due to water level difference, the water flows into the lock chamber. At this point, the load on the remaining lock gate(s) becomes exactly the same as the load on the failed lock gate. Thus, a large positive correlation should be accounted for. However, due to uncertainties in the resistance of the lock gates there is no full dependence. Based on these considerations, the correlation coefficient is assumed to be:

$$\rho(F_{i,WD}, F_{j,WD}) = 0.9$$

Based on Figure 7.3, small changes of this correlation coefficient noticeably influence the reliability of a system. This should be kept in mind when evaluating the results.

### 7.3. Fault tree of the flood defense system

#### 7.3.1. Relevant lockage procedures

The dike system that lays behind the lock complex has a retaining height of 10.00 m + NAP. Therefore, only water levels on the river Waal above 10.00 m + NAP are considered in the calculation of the flood probability. Hence, only two lockage procedures have to be considered in the fault tree analysis: stepped lockage and stepped retaining. For water levels on the river Waal above 15.25 m + NAP, overflow occurs. Failure of the lock gates is no longer relevant in that situation.

![Figure 7.6: Relevant lockage procedures for flooding](image)

#### 7.3.2. Flooding probability of the undamaged system

Only structural failure is considered as a cause of flooding. Structural failure may occur due to water level differences or due to ship collision. In practice, other failure modes may occur as well. In combination with
structural failure, these failure modes might result in flooding. An example of such a failure mode is non-
closure. Since the focus is on structural robustness, these failure modes are not considered in the calculations.

In the case of the stepped lockage procedure, structural failure of all three lock gates is required in order to
cause flooding. This procedure is active for water levels \(10.00 < h < 12.80\) m + NAP.

In the case of stepped retaining, only the Waal gate has to fail in order to cause flooding. This is due to the
fact that only the Waal gate is equipped with an extension piece. Once the Waal gate fails, the other lock
gates do not have sufficient retaining height and they will fail due to overflow. In addition, the probability of
structural failure of a lock gate due to ship collision is neglected for this procedure. This is justified by the
idea that navigation is obstructed during this procedure. Of course, in practice, there always is the very small
probability that a wayward ship collides with the lock gates.

**7.3.3. Flooding probability of the damaged system**

The fault tree from Figure 7.7 represents the flooding probability for the system without damage. For the
robustness calculation, we are interested in the flooding probability of the system with damage. This term is
part of the indirect risk, as was introduced in Chapter 4:

\[
P(\text{flood}) = P(\text{hazard}) \cdot P(\text{damage} | \text{hazard}) \cdot P(\text{flood} | \text{damage}) \cdot C(\text{flood})
\]

A complicating factor is the dependence between failure events. These dependencies are especially impor-
tant for the stepped lockage procedure. The failure events are structural failure of respectively the Waal gate,
the middle gate and the canal gate. Dependencies are accounted for by using correlation coefficients. These
have been introduced in Chapter 7.

Damage is modelled as failure of the Waal gate. Failure is either caused by water level difference or by a ship
collision. Because of dependencies, it is important that a distinction is made in the initial load that led to
failure of the Waal gate. Therefore the system failure probability will be calculated for damage caused by
water level differences and for damage caused by a ship collision.
Due to failure of the Waal gate, the fault tree from Figure 7.7 reduces to a more simplified fault tree. The Waal gate can be removed from the fault tree, as is done in Figure 7.8.

7.4. Fault tree of the navigation system

7.4.1. Relevant lockage procedures

For the analysis of delay probabilities due to structural failure, the normal lockage procedure and the stepped lockage procedure are relevant. For both lockage procedures, it can be stated that a short delay occurs when only one of the gates fails. When a second lock gate fails, a long delay occurs. For other cases, there already is a delay due to natural boundary conditions.

When only one lock gate fails, the other two gates are still available and can be used for lockage. Nonetheless, protocol requires that the middle gate is floated into the position of the Waal gate. This causes a short delay.

When two lock gates fail, this causes a long delay. All lockage procedures require at least 2 operational lock gates. Repair of the damaged gates or construction of new lock gates requires time spans in the order of weeks or months. In this special case, the presence of the western navigation lock strongly reduces the delay for navigation.

7.4.2. Delay probability of the undamaged system

In the undamaged situation, the navigation function fails when ships are delayed. This happens frequently because of low water levels on the river Waal. However, this research focuses on the structural robustness of the lock complex and therefore interest goes out to delays that are caused by structural failure. Figure 7.9 shows a fault tree with the series of events that result in delay of navigation.
7.4. Fault tree of the navigation system

7.4.3. Delay probability of the damaged system

Now, the damaged system is considered. Again, damage is modelled as loss of the Waal gate. Based on the fault trees from Figure 7.9, the probability of short delay for the damaged system is equal to 1. The probability of a long delay for the damaged system is given by the fault tree from Figure 7.10.

Structural failure of the Waal gate results in a short delay in any case. Although lockage might still be possible with the remaining two gates, the procedures require that there is always a gate present in the Waal head. At this position the extension piece has to be placed. To achieve this, the middle gate is floated into the position of the Waal head. This operation requires some time and navigation is delayed for some days. Therefore the probability of a short delay for the damaged system is equal to 1.

Once the middle gate has been floated into its new position in the Waal head, the possibility of a long delay is still present. In case one of the two remaining lock gates fails as well, lockage is no longer possible. Repair of the damaged gates or construction of new lock gates is required. This covers time spans of weeks or maybe even months. Hence, navigation is delayed for a long period.
7.5. Resulting system failure probabilities

7.5.1. Flooding probability

Table 7.2 shows the probability of flooding of the damaged system. Distinction is made between the two possible causes of damage. Appendix E provides an elaboration of the calculations that have been performed to obtain these failure probabilities for a value of $\beta_{\text{design}} = 3.00$. The procedure for other values of $\beta_{\text{design}}$ is exactly the same.

| $\beta_{\text{design}}$ | P (Flood | Damage due to WD) | P (Flood | Damage due to SC) |
|-------------------------|----------------------|----------------------|
| 2.00                    | 7.70E-01             | 4.37E-03             |
| 2.25                    | 7.43E-01             | 2.48E-03             |
| 2.50                    | 7.18E-01             | 1.43E-03             |
| 2.75                    | 6.92E-01             | 7.90E-04             |
| 3.00                    | 6.63E-01             | 4.07E-04             |
| 3.25                    | 6.33E-01             | 1.93E-04             |
| 3.50                    | 6.00E-01             | 8.40E-05             |
| 3.75                    | 5.65E-01             | 3.35E-05             |
| 4.00                    | 5.30E-01             | 1.23E-05             |
| 4.25                    | 4.94E-01             | 4.19E-06             |
| 4.50                    | 0.00E+00*            | 0.00E+00*            |
| 4.75                    | 0.00E+00*            | 0.00E+00*            |
| 5.00                    | 0.00E+00*            | 0.00E+00*            |
| 5.25                    | 0.00E+00*            | 0.00E+00*            |
| 5.50                    | 0.00E+00*            | 0.00E+00*            |
| 5.75                    | 0.00E+00*            | 0.00E+00*            |
| 6.00                    | 0.00E+00*            | 0.00E+00*            |

Figure 7.11: Flooding probability of the damaged system, original situation

The values from Table 7.2 are plotted in Figure 7.11. It can be noticed directly, that there is a significant jump in the interval $4.25 < \beta_{\text{design}} < 4.50$. In fact, the probability of flooding after damage becomes zero when $\beta_{\text{design}} \geq 4.50$.

In order to explain this jump, the information from Chapter 6 is required. The increased reliability requirement is modeled as an increase of the resistance. As long as the reliability requirement $\beta_{\text{design}} \leq 4.25$, the probability of structural failure due to ship collision is neglected because of its low probability of occurrence. Once $\beta_{\text{design}} \geq 4.50$, ship collision can no longer be neglected. In that case, the resistance has to be increased significantly to reduce the probability of structural failure due to ship collision. For clarity, Figure 6.12 is given again in Figure 7.12.

Apparently, the resistance increases with such magnitude that the probability of structural failure due to water
level difference reduces to (approximately) zero. This explains why there is a jump in the flooding probability after damage due to water level difference.

The jump in the flooding probability after damage due to ship collision, has the same explanation. However, there is an additional explanation that has to do with assumed correlation coefficients. Due to the assumption of a negative correlation coefficient between two ship collision coefficient, the probability of two ship collisions at the same moment is extremely small. It turns out that, when damage has occurred due to ship collision, the probability of flooding is still governed by failure due to water level differences. Hence, the jump in the flooding probability after damage due to ship collision can be explained by the jump in required resistance as well.

### 7.5.2. Delay probability

Similar to the flooding probability, Table 7.3 (next page) shows that the probability of delay of the damaged system drops when the reliability requirement is \( \beta_{\text{design}} \geq 4.50 \). The explanation is given by the jump in the required resistance, identical to the case of flooding.

Values from Table 7.3 are plotted in Figure 7.13. The curves look very similar to the curves of the flooding probability (Figure 7.11). Again, the system failure probabilities differ a lot per cause of damage. Also these differences can explained in the exact same way as they were explained for flooding.
7. System failure probabilities

Table 7.3: Long delay probability per year of the damaged system. *Due to the fact that \( P(\text{Damage} | \text{WD}) \approx 0 \).

| \( \beta_{\text{design}} \) | \( P(\text{Long delay} | \text{Damage due to WD}) \) | \( P(\text{Long delay} | \text{Damage due to SC}) \) |
|---|---|---|
| 2.00 | 9.21E-01 | 2.54E-02 |
| 2.25 | 9.03E-01 | 1.40E-02 |
| 2.50 | 8.84E-01 | 7.26E-03 |
| 2.75 | 8.63E-01 | 3.57E-03 |
| 3.00 | 8.40E-01 | 1.66E-03 |
| 3.25 | 8.16E-01 | 7.27E-04 |
| 3.50 | 7.90E-01 | 3.01E-04 |
| 3.75 | 7.63E-01 | 1.17E-04 |
| 4.00 | 7.35E-01 | 4.30E-05 |
| 4.25 | 7.06E-01 | 1.49E-05 |
| 4.50 | 0.00E+00* | 6.64E-14 |
| 4.75 | 0.00E+00* | 2.06E-15 |
| 5.00 | 0.00E+00* | 5.32E-17 |
| 5.25 | 0.00E+00* | 1.14E-18 |
| 5.50 | 0.00E+00* | 2.01E-20 |
| 5.75 | 0.00E+00* | 2.96E-22 |
| 6.00 | 0.00E+00* | 3.60E-24 |

In contrast to flooding, the probability of a long delay after damage caused by ship collision doesn’t turn to zero for large values of \( \beta_{\text{design}} \). This is due to the fact that structural failure of only two gates is required. Due to the assumed negative correlation, failure probabilities are small but non-zero. However, probabilities are sufficiently small to neglect them.

7.5.3. System failure probability per cause of damage

Figure 7.14 illustrates the probability of system failure per cause of damage. It shows that the probability of failure of both primary functions behaves similarly. As expected, for both causes of damage, the system failure probability is lowest for the flood defense function. This is explained by the requirement that all lock gates have to fail. In contrast to the navigation system, which also fails when one lock gate remains in tact.

From the figure, it can also be noticed that the system failure probabilities are significantly lower when damage is caused by ship collision. This is explained by the assumed negative correlation coefficient for two ship collision events.

![Failure probability, after damage due to WD](a)

![Failure probability, after damage due to SC](b)

Figure 7.14: System failure probabilities per cause of damage.
The failure probability of a system can be analysed by constructing a fault tree that contains (series of) events that may lead to failure. Within these fault trees it is important to distinguish between series systems (OR-gates) and parallel systems (AND-gates). In addition, it is important to account for dependencies between the events. This is done by estimation of correlation coefficients.

For the case study, the lockage procedures have a large influence on the fault trees. Because the definition of failure is different for the flood defense function and the navigation function, separate fault trees have to be constructed for both functions. Correlations are estimated for simultaneous failure of multiple lock gates. In short, after failure of a lock gate due to water level differences increases it is likely that the next lock gate fails as well. The opposite holds for ship collision (see Section 7.2.2). As a result, the probability of system failure is largely dominated by failure due to water level differences. System failure, solely due to ship collision, is almost impossible.

The results from the case study show that the failure probability of the system decreases when the required reliability of the structural elements is increased. When the requirement becomes $\beta_{\text{design}} \geq 4.50$, the probability of failure rapidly decreases. This is explained by the jump in the required resistance that is caused by the shift of the governing hazard (see Chapter 6).

When ship collision is considered as the governing load for the structural elements, the elements are designed so strong that the probability of damage due to water level differences is practically zero. Due to the estimated correlations, the probability of system failure becomes approximately zero.

Hence, the results of the case study are significantly influenced by the increase that is required for the resistance. As explained in Chapter 6, this jump might be less abrupt in practice. The case study, however, still gives valuable information on the behaviour of the system. More general, it can be stated that increasing the reliability requirement $\beta_{\text{design}}$ results in a more reliable system. When considering multiple hazards, a sudden increase in the required resistance of the elements results in a sudden increase of the system's reliability.
Results

This chapter consists of roughly 2 parts. The first part presents the calculated results. First, the direct risks (8.1) and indirect risks (8.2) are discussed. Then, the resulting robustness indices follow in Section 8.3.

Part two analyses the results. Section 8.4 examines the sensitivity of the results with respect to the assumed consequences. Also, an additional calculation of the hypothetical scenario in which the middle gate would not be present.

8.1. Direct risks

As defined in Chapter 4, direct risks are those risks that can be related to damage of the system. Within this research, direct risks are those risks related to failure of the Waal gate. The direct risk is defined as:

\[ R_{direct} = \sum P(H) \cdot P(D|H) \cdot C(D) \]

- \( P(H) \): Hazard occurrence probability (Chapter 5)
- \( P(D|H) \): Damage probability given that the hazard occurs (Chapter 6)
- \( C(D) \): Consequence of damage (Chapter 5)

Using the values from the given chapters, the direct risks can be calculated per initial hazard. Values are presented in Table 8.1 and Figure 8.1.

From Figure 8.1 it can be clearly seen that the direct risk decreases gradually with increasing reliability requirement. This is in line with expectations: the Waal gate is assumed to be designed such that its failure probability corresponds directly to the reliability requirement.
Table 8.1: Results: direct risks

<table>
<thead>
<tr>
<th>$\beta_{\text{design}}$</th>
<th>Water difference</th>
<th>Ship collision</th>
<th>Total direct risk</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.00</td>
<td>£ 227,501</td>
<td>£ 44</td>
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<td>£ -</td>
<td>£ 0</td>
<td>£ 0</td>
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</table>

8.2. Indirect risks

Indirect risks are those risks associated with system failure. Within this research, two systems (functions) of which Sluis Weurt is an element are considered: the flood defense system and the system of main waterways. The general formula for indirect risks is given as:

$$R_{\text{indirect}} = \sum P(H) \cdot P(D|H) \cdot P(F|D) \cdot C(F)$$

- $P(H)$: Hazard occurrence probability (Chapter 5)
- $P(D|H)$: Damage probability given that the hazard occurs (Chapter 6)
- $P(F|D)$: System failure probability given that damage occurs (Chapter 7)
- $C(F)$: Consequence of system failure (Chapter 5)

Compared to direct risks, the indirect risks are hard to summarize in a single table or graph. Therefore the indirect risks are presented per function.

8.2.1. Flooding

Failure of the flood defense function has been discussed in Chapter 7. It showed that the flooding probability, amongst others, depends on the water level and the operational lockage procedure. Calculation of the flood risk can be found in Appendix F, the results are presented in Table 8.2 and Figure 8.2.
The graph shows that the flood risk gradually decreases for an increasing reliability requirement. In correspondence with the results from Chapter 7, the flood risk becomes zero for a reliability requirement $\beta_{\text{design}} \geq 4.50$.

As was explained in Section 7.5 this is caused by the fact that the event of ship collision suddenly becomes the governing load. As a result, there is a jump in the resistance of the lock gates. The probability that these lock gates fail due to water level differences becomes approximately zero. It was also explained that failure of multiple lock gates due to a ship collision has an extremely low probability.

### 8.2.2. Navigation delay

Failure of the navigation function has been discussed in Chapter 7. It showed that a distinction should be made between situations with a short delay and situation with a long delay. Both sub-consequences are incorporated in the risks that are presented in Table 8.3 and Figure 8.3. Calculation of the delay risk can be found in Appendix F.

---

**Table 8.2: Results: indirect risk per year, flooding**

<table>
<thead>
<tr>
<th>$\beta_{\text{design}}$</th>
<th>Water difference</th>
<th>Ship collision</th>
<th>Total flood risk</th>
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Figure 8.3: Results: indirect risk, navigation delay
Table 8.3: Results: indirect risk, navigation delay

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<td>5.00</td>
<td>€ -</td>
<td>€ 0</td>
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</tr>
<tr>
<td>5.25</td>
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</tr>
<tr>
<td>5.50</td>
<td>€ -</td>
<td>€ 0</td>
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</tr>
<tr>
<td>5.75</td>
<td>€ -</td>
<td>€ 0</td>
<td>€ 0</td>
</tr>
<tr>
<td>6.00</td>
<td>€ -</td>
<td>€ 0</td>
<td>€ 0</td>
</tr>
</tbody>
</table>

From Figure 8.3 it can be seen that the risk of delay due to a ship collision can be subdivided into two stages. The first stage is for the interval $2.00 \leq \beta_{\text{design}} < 4.50$, the second stage for the interval $4.50 \leq \beta_{\text{design}} < 6.00$. These intervals coincide with the intervals in which respectively the water level difference and ship collision are governing. Apparently, reducing the reliability requirement is most effective when this is done for the governing load. Note that water level differences on itself, still contributes the most to the risk of navigation delay.

8.3. Robustness indices

In Chapter 4, the definition of robustness from different perspectives has been discussed. The definition as it is used in the Eurocodes has been used for this research and is given as:

"Robustness is the ability of a structure to withstand extreme events or the consequences of human error, without being damaged to an extent disproportionate to the original cause"

Quantification of this robustness definition can be done by quantifying the proportionality between cause and consequence. The quantification method as proposed by the Joint Committee on Structural Safety considers a robustness index that can be written as the ratio between direct and indirect risk:

$$ I_{\text{rob}} = \frac{R_{\text{indirect}}}{R_{\text{indirect}} + R_{\text{indirect}}} $$

First, the robustness of the structural system as a whole is considered. All functions and their risks are considered together. Hence, the indirect risks of flooding and navigation delay are added up. Subsequently, the robustness indices are calculated per function. A closer look will be taken to the contribution of each hazard to the robustness.

8.3.1. Total system

First, the combined risk for both the flood defense system and the navigation system is calculated. For each value of the reliability requirement the direct risk, total indirect risk and robustness indices are presented in Table 8.4. The robustness indices are plotted in Figure 8.4.
Table 8.4: Results: robustness indices of the combined system (flood defense & navigation).

<table>
<thead>
<tr>
<th>$\beta_{\text{design}}$</th>
<th>Direct risk</th>
<th>System failure risk</th>
<th>Robustness index</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.00</td>
<td>€ 227,545</td>
<td>€ 22,765,609</td>
<td>0.00990</td>
</tr>
<tr>
<td>2.25</td>
<td>€ 122,407</td>
<td>€ 12,425,306</td>
<td>0.00975</td>
</tr>
<tr>
<td>2.50</td>
<td>€ 62,130</td>
<td>€ 6,890,949</td>
<td>0.00894</td>
</tr>
<tr>
<td>2.75</td>
<td>€ 29,839</td>
<td>€ 3,683,220</td>
<td>0.00804</td>
</tr>
<tr>
<td>3.00</td>
<td>€ 13,543</td>
<td>€ 1,840,097</td>
<td>0.00731</td>
</tr>
<tr>
<td>3.25</td>
<td>€ 5,814</td>
<td>€ 846,812</td>
<td>0.00682</td>
</tr>
<tr>
<td>3.50</td>
<td>€ 2,370</td>
<td>€ 358,013</td>
<td>0.00658</td>
</tr>
<tr>
<td>3.75</td>
<td>€ 928</td>
<td>€ 139,284</td>
<td>0.00661</td>
</tr>
<tr>
<td>4.00</td>
<td>€ 360</td>
<td>€ 49,954</td>
<td>0.00716</td>
</tr>
<tr>
<td>4.25</td>
<td>€ 150</td>
<td>€ 16,581</td>
<td>0.20646</td>
</tr>
<tr>
<td>4.50</td>
<td>€ 34</td>
<td>€ 0</td>
<td>0.98643</td>
</tr>
<tr>
<td>4.75</td>
<td>€ 10</td>
<td>€ 0</td>
<td>0.98643</td>
</tr>
<tr>
<td>5.00</td>
<td>€ 3</td>
<td>€ 0</td>
<td>0.98643</td>
</tr>
<tr>
<td>5.25</td>
<td>€ 1</td>
<td>€ 0</td>
<td>0.98643</td>
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<tr>
<td>5.50</td>
<td>€ 0</td>
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<tr>
<td>6.00</td>
<td>€ 0</td>
<td>€ 0</td>
<td>0.98643</td>
</tr>
</tbody>
</table>

The robustness index shows a discontinuity when the reliability requirement becomes larger than 4.50. The underlying mechanism is exactly the same as the mechanism in Chapters 6 and 7: ship collision becomes the governing load which results in a significant increase of the required resistance. As it appears, the direct risk becomes larger than the indirect risk. The result is a high robustness index. For lower values of the reliability requirement, this was exactly the other way around, which results in a low robustness index.

The strong reduction of the system failure probability (indirect risk) was already noticed in Section 7.5. The failure probability of the damaged system is dominated by the contribution of water level differences. This contribution approaches zero when the required resistance suddenly increases significantly.

Ship collision, on the other hand, doesn’t contribute much to the probability of failure of the damage system. This is due to the assumed negative correlation coefficient for multiple failing lock gates due to ship collision. This means that also an increase or decrease of the failure probability due to ship collision will be hard to notice.

Another noticeable fact about the results is the fact that the robustness index initially drops slightly. This is illustrated in the enlarged plot of Figure 8.4. However, the differences are sufficiently small to assume that direct and indirect risks are in balance as long as $\beta_{\text{design}} \leq 4.50$. 
8.3.2. Individual systems

Figure 8.5 shows the robustness of the flood defense system. The direct risk of these robustness indices corresponds to loss of the Waal gate, which is exactly equal for both the flood defense system and the navigation system. The difference is in their indirect risks. The robustness indices of the flood defense system are shown in Figure 8.5. These are almost exactly equal to the robustness indices of the total systems.

Robustness indices of the navigation system are shown in Figure 8.6. These robustness indices are clearly higher than those of the flood defense system. This can be explained by the fact that the risk of navigation delay is significantly lower than the risk of flooding. Even though the probability of navigation delay is higher, the consequences are lower compared to flooding (see Chapter 7).

Comparing Figures 8.5 and 8.6 to Figure 8.4, it seems that robustness of the total system is governed by the robustness of the flood defense system. The risks of navigation delay are negligible compared to the risks of flooding. The jump that can be seen in Figures 8.5 and 8.6 is caused by the discontinuity in the direct and indirect risks. See Chapters 6 and 7 for an explanation.

8.3.3. Cause of damage

Per system, the risks are calculated for two possible causes of damage: water level difference and ship collision. The probability of failure of the damaged system, depends on the initial hazard that caused this damage. The robustness indices in Figures 8.5 and 8.7 are based on both hazards contributing to the risk. Figure 8.7 shows the robustness indices with respect to the considered hazards.

First, it should be mentioned that the direct risks are different for each hazard. For both hazards, the direct consequence is equal: loss of the lock gate in the Waal head. Yet, due to the difference in the assumed correlation coefficients, the failure probability of the damaged system is different.

Water level difference is assumed to have a large and positive correlation coefficient. Hence, the probability that the damaged system fails is relatively high. Ship collision on the other hand is assumed to have a negative
8.4. Analysis of the results

8.4.1. Sensitivity analysis: consequences of Waal gate failure

In Chapter 5 the costs of replacing the Waal gate have been estimated at €10 million. This estimation is based on the idea that a completely new lock gate has to be designed, constructed and installed. In practice, consequences of damage might be lower.

For this reason it is examined how the robustness indices are effected by for lower estimates of the direct consequences. Based on the general robustness formula, it can be expected that lower values of the direct consequence result in lower values of the robustness index. In addition to the initial estimation, the robustness index is calculated for a direct consequence equal to €5 million and €1 million.

Figure 8.8 shows that the robustness indices are lower indeed. For the flood defense system, however, the absolute difference is relatively small. This is due to the fact that the (indirect) risks of flooding are significantly larger for all estimations of the direct consequence. In case of the navigation system, the risk of damage and the risk of system failure are more close to one another. As a result, reduction of the direct consequences has a more significant impact on the robustness index.
8.4.2. Sensitivity analysis: consequences of flooding

Consequences of flooding have been discussed in Chapter 5. A distinction in consequences has been made, based on the inundation depth that is present in case of flooding at specific water level intervals. In practice, consequences of failure might be reduced. For example, consequences are reduced by emergency measures that aim to reduce the inflow of water (e.g. sandbags, box barrier). In addition, the past few years there has been more attention for a multi-layer safety approach [30]. This has brought more focus on flood proof infrastructure and evacuation measures.

Based on the above considerations, the actual damage due to flooding could be lower than is assumed in the calculations. For this reason it is examined how the robustness indices change for lower estimates of the flood consequences. It is to be expected that robustness increases for lower estimates of indirect consequences. Reductions of 25% and 50% are considered.

As can be seen from Figure 8.9, the robustness indeed increases. However, the increment of the robustness index is very limited. This is explained by the fact that there still is a large gap between the risk of damage and flood risk. A reduction of the flood risk by 50% is already optimistic and further decrease of the consequences would be unrealistic. A reduction of the flooding probability might be a solution to further reduce flood risk.

8.4.3. Sensitivity analysis: consequences of navigation delay

In Chapter 6 the consequences have been estimated for navigation delay as well. These consequences are limited due to the presence of the western navigation lock. At many locations in the Netherlands, such an additional navigation lock isn’t present. It can be expected that the robustness of the navigation system is lower in the case there is only a single lock. It is examined to what extent the robustness of the navigation system is increased by the presence of the Western lock.

In case of no Western lock at Weurt (or failure of the Western lock), failure of the Eastern lock means that ships are not able to pass. In this case, another route is available which was used before construction of the...
8.4. Analysis of the results

Meuse-Waal canal in 1927. It is a detour of approximately 100 kilometer through the St. Andries canal. Based on an average sailing speed of 10 kilometers this would mean that on average, each ship is delayed with 10 hours. In the original estimations, the maximum delay of a ship was estimated to be only 1 hour.

Figure 8.10 shows that the robustness of the navigation system is significantly decreased by the increased consequences of failure. Apparently, the gap between the (direct) risk of failure of the Waal gate and the (indirect) risk of delay is relatively small. As a result, adjusting the indirect consequences has a significant impact on the robustness index.

8.4.4. Sensitivity analysis: effect of a 3rd gate

The presence of the middle lock gate improves the system reliability of the navigation lock: it requires an additional event in the fault tree. It is expected that this also results in a higher robustness index. But is yet unknown what the magnitude of this effect is.

It is now examined what the robustness index of both the flood defense system and the navigation system would be in the case that the lock only has 2 gates. In practice, this would result in larger water level differences over the lock gates and different design values of the resistance and thus of the damage probabilities. For simplicity of the calculation, the same resistance values and failure probabilities are used as in the case of 3 lock gates.

In case of a navigation lock with only 2 lock gates, the probabilities of system failure are larger. This is due to the fact that one gate less has to collapse to cause system failure. Figure 8.11 shows the robustness indices for the situation with 3 lock gates and the situation with 2 lock gates.

![Robustness: flooding without 3rd lock gate](image)

![Robustness, navigation system without 3rd lock gate](image)

Figure 8.11: Response of the results to a reduction of the number of lock gates. Note the different scales of the vertical axes.

The graphs from Figure 8.11 show that the robustness is indeed larger for a system with 3 lock gates. For the flood defense system, the difference in robustness is not so large. Even though the (indirect) risks of flooding change, in both situations the flood risks are significantly larger than the (direct) risk of losing the Waal gate.

Regarding the navigation system, it can be seen that the robustness index for a system with 2 lock gates is not effected when the reliability requirement of a lock gate is increased. This is not surprising because, in this case, damage equals system failure. The navigation lock is unable to allow passage of ships when only one lock gate is available.
8.5. Summary & conclusion

The level of robustness is expressed as an index by using Formula 8.2. The first two sections of this chapter have discussed the direct risks and indirect risks for multiple values of $\beta_{\text{design}}$. Because the consequences have been related to a fixed definition of failure, the risks are linearly related to the system failure probabilities that have been found in Chapter 7.

\begin{equation}
 I_{\text{rob}} = \frac{R_{\text{indirect}}}{R_{\text{indirect}} + R_{\text{indirect}}}
\end{equation}

For the case study, the robustness indices have been calculated for the individual functions as well as for the total system. In addition, robustness indices have been calculated with respect to individual hazards. Subsequently, a sensitivity analysis has been performed to examine the influence of the involved parameters.

First, the robustness of the total system is discussed. For low values of the reliability requirement ($\beta \leq 4.25$), the indirect risks are significantly larger than the direct risks and the total system has a low robustness index. For these values, the robustness initially drops slightly and then increases again. For higher values of the reliability requirement ($\beta \geq 4.50$), there is a jump in the robustness index. The indirect risks become significantly smaller than the direct risks, which results in a high level of robustness. The jump is explained in accordance with Chapters 6 and 7.

When splitting the functions, it is found that the robustness of the total system is governed by the robustness of the flood defense system. The robustness indices are almost equal for the total system and the flood defense system.

The robustness of the navigation system, on the other hand, shows a different pattern. These robustness indices are all relatively high: $I_{\text{rob}} \geq 0.73$ for all values of $\beta_{\text{design}}$. In any case, the indirect risks of system failure are relatively small compared to the direct risks. Hence, the navigation system is considered to be robust. This is caused by the lower consequences of failure.

Figure 8.7 shows the contribution of the individual hazards. It is shown that the robustness indices of the individual systems are determined by the governing loads. For values $\beta \leq 4.25$, the robustness of the system is given by the robustness with respect to water level differences. For values $\beta \geq 4.50$, the robustness of the system is given by the robustness with respect to ship collision.

Finally, a sensitivity analysis has been performed. The most interesting results are found when analysing the effect of redundant elements. In the hypothetical case of a single navigation lock, the robustness of the navigation system would be significantly lower. The (redundant) Western navigation lock ensures a low probability of total system failure (one could also argue that it reduces the consequences of failure).

Another interesting result is the fact that the robustness index drops significantly for low consequences of damage. In other words, an expensive replacement of the lock gate results in a more robust system than a cheap replacement. This seems logical, since high quality engineering (low probability of failure) is more expensive in general. Nonetheless, this result can not be used to justify large investments.

In general, it can be concluded that increasing the reliability requirement of the individual structural elements does influence the robustness of the system. Normally, the effect on the robustness index is small and can be either positive or negative. Only for the case in which the system reliability is significantly affected by increasing $\beta_{\text{design}}$, the robustness shows a rapid positive increase.
III

Evaluation
Conclusions and recommendations

9.1. Conclusions

This section discusses the conclusions that can be drawn from the performed research. The objective of the research was formulated in the introduction as:

"The objective of this research is to quantify the effect of the reliability requirement for structural elements on the level of robustness of a navigation lock that is part of the primary flood defence system."

First, the main conclusions that relate to the research objective are presented. Subsequently, the supporting research questions are briefly answered.

9.1.1. Main conclusions

1. Increasing the reliability requirement of structural elements doesn't automatically increases the level of robustness

Increasing the reliability requirement of individual structural elements ($\beta_{\text{design}}$), does indeed influence the robustness of the structural system. However, the effect is not necessarily positive. By increasing $\beta_{\text{design}}$, both the (direct) risk of damage and (indirect) risk of system failure are reduced. Whether robustness increases, depends on the relative change of both risks.

2. A substantial increase of robustness requires a reduction of the probability of failure, after damage

The case study has shown a substantial increase of the robustness index. The increase has multiple causes that are specific for the case. It is not necessarily representative for other cases. Nonetheless, it shows that reduction of the (indirect) risk of system failure after damage is an effective strategy to increase robustness. In essence, consequences of system failure are fixed. Therefore, the main strategy to increase robustness is reduction of the probability of system failure after damage.

3. Adding redundant elements to the design increases the level of robustness

Redundant elements in the case study are an additional lock gate and an additional navigation lock. The results show that these redundant elements have a positive effect on the robustness. The benefit of redundant elements, is that they do not reduce the (direct) risk of damage. Only the (indirect) risk of system failure is reduced. Hence, the effect on the robustness index is always positive. Therefore,
adding redundant elements to the design seems to have an advantage over increasing the reliability requirement.

4. **Robustness indices have to be calculated individually for each function**

It has been shown that a single structure has a different robustness index for each function, see Figure 9.1. When risks of all functions are combined, the resulting robustness index is dominated by the function with the largest risk. For example, results from the case study show that there is hardly any difference between robustness indices of the flood defense system and the total system.

![Figure 9.1: Robustness indices of the individual systems (Chapter 8).](image)

**Case study ‘Sluis Weurt’**

The navigation lock near Weurt has been used to evaluate the influence of the reliability requirement for structural elements. To do so, the probability of occurrence and the magnitude of the loads have been analysed. In addition, the consequences of damage and the consequence of failure are based on the characteristics of the case study. However, the actual resistance of the structural elements has not been assessed. As a result, it is not possible to make a statement about the actual probabilities of damage (failure of a lock gate) and system failure (e.g. flooding, navigation delay). Hence, it can not be stated whether ‘Sluis Weurt’ is a robust system or not.

**9.1.2. Research questions**

1. **What methods are used to derive the structural reliability requirement?**

*Chapters 2 and 3.*

The Netherlands has two laws that set reliability requirements for structures that are part of the primary flood defense: the Water Law and the Law on Buildings. There are two main differences:

1. The Waterlaw sets a reliability requirement for the total structure. Subsequently, this requirement is directly applied on the individual elements. The Law on Buildings directly gives a reliability requirement for the individual structural elements.

2. The reliability requirement in the Waterlaw is based on the consequences of flooding at the specific location of the structure. The reliability requirement in the Law on Buildings is based on more general consequence classes.
2. How is robustness defined and what parameters are involved in quantification of robustness?

Chapter 4.

The Joint Committee on Structural Safety proposes a robustness index that is based on the ratio of direct risks and indirect risks:

\[ I_{rob} = \frac{R_{direct}}{R_{direct} + R_{indirect}} \]  
\[ R_{direct} = \sum P(H) \cdot P(D|H) \cdot C(D) \]  
\[ R_{indirect} = \sum P(H) \cdot P(D|H) \cdot P(F|D) \cdot C(F) \]

- \( P(H) \) Occurrence probability of a considered hazard
- \( P(D|H) \) Probability that occurrence of the hazard results in damage to the navigation lock
- \( P(F|D) \) Probability that occurrence of damage results in failure of the parent system
- \( C(D) \) Costs that are involved in repair of the navigation lock
- \( C(F) \) Consequences of failure of the parent system (e.g. flood, navigation delay)

3. What is the relation between robustness and the reliability requirement?

Chapter 4.

There is a strong relation between the reliability requirement and the robustness index. Important parameters are:

- \( P(D|H) \) Probability that occurrence of the hazard results in damage to the navigation lock
- \( P(F|D) \) Probability that occurrence of damage results in failure of the parent system

4. What is the effect of increasing the reliability requirement on the level of robustness?

Chapters 6 up to 8

First of all, the case study has shown that increasing the reliability requirement (\( \beta_{design} \)) does not have a straightforward effect on the robustness index. Because both the (direct) risk of damage and the (indirect) risk of system failure are reduced by an increase of \( \beta_{design} \), the robustness index can also decrease. This is shown in the right hand of Figure 9.2

The results of the case study also show a sudden increase of the robustness index in at \( \beta_{design} = 4.25 \) (left hand of Figure 9.2). A closer look into the underlying mechanisms, reveals that the jump is caused because of the fact that the probability of system failure is significantly decreased (approximately zero). The decrease has multiple causes that are specific for the case. It is not necessarily representative for other cases. Nonetheless, it shows that reducing the probability of system failure after damage is very effective to increase robustness.
A third conclusion that can be drawn, is that increasing $\beta_{\text{design}}$ is more effective for systems with low consequences of failure. This is substantiated by the difference in robustness indices of the flood defense system (high consequence) and the navigation system (low consequence). This can be clearly seen in Figure 9.1.

Note 1: in the case study, it is assumed that the reliability requirement has to be satisfied by increasing the resistance of the structural elements. When ship collision becomes the governing load, the increase of the required resistance is almost a factor 10. In practice, it might be more economical to invest in a collision protection in front of the lock gate.

5. What is the effect of adjusting other parameters that are involved in quantification of robustness?

Chapter 8

The most noticeable effect on the robustness index is caused by the presence of redundant elements. Redundant elements in the case study are an additional lock gate and an additional navigation lock. Results show that the redundant elements have a positive influence on the robustness index, see Figure 9.3.

The benefit of redundant elements is that they do not influence the direct risk. Only the probability of failure is reduced. As a result, adding redundant elements always has a positive effect on the robustness index.

Reduction of the consequences of damage seems logical from a risk perspective. However, reducing damage consequences also means that robustness decreases. This is caused by the fact that it increases the relative contribution of (indirect) risk to the robustness index (see Formula 9.1).

Lower estimations of the consequences of system failure result in higher robustness. This is also logically explained by through Formula 9.1. By decreasing the consequences of system failure, the indirect risk decreases as well. Hence, the denominator of the index reduces and the index increases.
9.2. Recommendations

9.2.1. Application of robustness quantification

1. **Robustness measures should focus on reducing the failure probability of the damaged system**

   Reduction of the failure probability after damage, is the most effective measure to increase robustness. Even though reduction of the damage probability results in a lower risk of system failure (e.g. flooding, navigation delay), it does not necessarily increase the robustness.

   *Practical recommendation:* To increase robustness, it is more effective to add redundant elements to the design instead of increasing the reliability requirement of the individual structural elements.

2. **Increasing robustness should be done within the perspective of current regulations**

   The results from the case study show that risks of system failure (e.g. flooding, navigation delay) are extremely small. These risk levels are significantly lower than the acceptable risk levels that have been used for guidelines such as the Wettelijk Beoordelings Instrumentarium (WBI2017, [6]). Risk levels that have been used in the WBI2017 are well substantiated and based on cost-benefit analysis. Very good arguments are required to invest in even lower risk levels.

   *Practical recommendation:* Reduction of the system failure risk (e.g. flooding, navigation delay) should be limited to commonly accepted risk levels.

3. **Robustness of the total system is not equal to robustness of the structural system**

   Robustness of a structural system is determined by more failure modes than just structural failure. For example, electrotechnical or mechanical failure (e.g. non-closure).

9.2.2. Further development of the applied framework

1. **Additional case study research**

   The case study on 'Sluis Weurt' shows that the robustness index is influenced by specific characteristics of the navigation lock. Additional case studies should be performed to obtain more knowledge about the general effect on the level of robustness of different measures. Specific recommendations to improve the case study results are given in 9.2.3

2. **A method should be developed to express robustness in terms of monetary value**

   At this point, robustness is a dimensionless number. An expression of robustness in terms of money will help to justify investments in measures to improve robustness. Risks for example, can be expressed in terms of money. This allows for an easy cost-benefit comparison of multiple measures to control the risk.

3. **The effect of maintenance plans and inspections on robustness should be examined**

   Maintenance plans and inspections are measures that focus on identifying possible damage. Timely identification of damage, reduces the probability of failure of the structure as a whole. Based on the conclusions, these measures promise to be very effective in increasing the level of robustness. The extended framework from Chapter 4 can be used.
4. **Further research should be done on the influence of material behaviour**

In the performed case study, damage has been modeled as a sudden loss of the critical elements. This means that the actual material behaviour of the structural elements is completely neglected. In practice, damage will be more complex. The load causes an initial breach. The characteristics of the breach and structural elements significantly influence both direct and indirect risk. The choice of material therefore influences the level of robustness.

**9.2.3. Improvements of case study results**

1. **An accurate determination of the loads is required**

An important characteristic of the performed case study is the large gap between the forces that are caused by water level differences and the forces that are caused by ship collision. Forces caused by water level differences are relatively small due to characteristics of the case. Forces caused by ship collision are relatively large due to made assumptions. The large difference in forces has great impact on the resulting robustness indices. A more accurate determination of the loads is required to improve the results of the case study.

2. **Calculate robustness indices for larger reference periods**

The current results are based on failure probabilities and risks per year. Robustness indices should be based on probabilities and risks within the lifetime of a structure (typically a design lifetime of 100 year).

3. **Formulate clear definitions of ‘damage’ and ‘system failure’**

Robustness remains a matter of definitions. Before calculations are started, a clear distinction is required between damage and system failure. A Failure Mode Effects & Criticality Analysis (FMECA) has proven to be a convenient tool to filter out the most important damage states. Fault Tree Analysis (FTA) is a good method to evaluate the probability of system failure. However, it requires a clear definition of the top event ‘system failure’.
9.3. Reflection on the applied framework

First of all, the used framework has the advantage that it can be applied on other types of structures as well. For example, the weir at Grave that was mentioned in the introduction.

<table>
<thead>
<tr>
<th>Applicability on other structures (weir at Grave):</th>
</tr>
</thead>
<tbody>
<tr>
<td>The incident with the weir at Grave, was caused by a ship collision. When focusing on ship collision as the only hazard, the equation of the robustness index reduces to:</td>
</tr>
</tbody>
</table>

\[
I_{\text{rob}} = \frac{R_{\text{dir}}}{R_{\text{dir}} + R_{\text{ind}}}
\]

\[
I_{\text{rob}} = \frac{P(H) \cdot P(D|H) \cdot C(D)}{P(H) \cdot P(D|H) \cdot C(D) + P(H) \cdot P(D|H) \cdot P(F|D) \cdot C(F)}
\]

\[
I_{\text{rob}} = \frac{C(D)}{C(D) + P(F|D) \cdot C(F)}
\]

In the case of the weir at Grave, the consequences of damage were quite large. But also the indirect consequence of failure and the probability of failure after damage were quite high. The following values are estimated:

\[
P(F|D) \approx 1
\]

\[
C(D) \approx €3,000,000.00
\]

\[
C(F) \approx €30,000,000.00
\]

The resulting robustness index, with respect to ship collision, can easily be calculated:

\[
I_{\text{rob}} = 0.09
\]

It can be concluded that the Weir at Grave is not a robust structure.

It has also been shown that the framework has disadvantages:

1. The concept of robustness remains a matter of definition. Other interpretations of robustness are equally possible. The same problem applies to the distinction between direct risk and indirect risk.

2. The robustness index returns a dimensionless number that cannot be expressed in economic value. Two systems, with completely different characteristics, may have the exact same robustness index. This makes it complicated to compare possible measures to increase robustness.

3. It doesn’t tell when a structure is sufficiently robust. Hence, no limit is set to investments in risk reduction measures.

Based on the above concerns, recommendations have been given to improve the framework. In particular the fact, that robustness can not be expressed in terms of economical value, makes it difficult to see the benefit of implementing robustness.

Increasing robustness, mainly comes down to increasing the reliability of the structural system. However, it doesn’t tell what the reliability of the system should be. The risk based approach that is used in new safety standards for the flood defense system in the Netherlands, also aims to increase reliability of the structural system. But a clear difference is that this risk-based approach does tell what the reliability should be.

Deviating from the risk-based reliability requirement, does require some good arguments. These can not (yet) be provided by the robustness framework. Perhaps, for now, a structure that satisfies a (risk-based) reliability requirement should be considered sufficiently robust.
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Appendices
Overview of parent systems

This appendix gives an overview of the dike rings in the Netherlands. A large amount of information originates from the VNK2 project [36] which assessed the consequence and probability of failure of 58 dike rings. In addition, characteristics of the system of main waterways is presented.

- Figure A.1 gives an overview of the VNK2 dike rings
- Figure A.2 gives an overview of all dike rings in the Netherlands
- Figure A.3 gives an overview of dike ring 41 "Land van Maas en Waal"
- Figure A.4 gives an overview of the main waterways in the Netherlands
- Figure A.5 gives an overview of vessel dimensions per CEMT class
### A. Overview of parent systems

#### Figure A.1: Dike rings assessed by VNK2 project [36]

<table>
<thead>
<tr>
<th>Nr.</th>
<th>Name</th>
<th>Type A defenses (km)</th>
<th>Number hydraulic structures</th>
<th>Area (ha)</th>
<th>Population size</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Schiermonnikoog</td>
<td>13.1</td>
<td>1</td>
<td>880</td>
<td>1.020</td>
</tr>
<tr>
<td>2</td>
<td>Ameland</td>
<td>36.7</td>
<td>3</td>
<td>3.250</td>
<td>3.560</td>
</tr>
<tr>
<td>3</td>
<td>Terschelling</td>
<td>27.7</td>
<td>3</td>
<td>2.300</td>
<td>4.700</td>
</tr>
<tr>
<td>4</td>
<td>Vlieland</td>
<td>2.3</td>
<td>2</td>
<td>281</td>
<td>1.100</td>
</tr>
<tr>
<td>5</td>
<td>Teylingen</td>
<td>26.0</td>
<td>10</td>
<td>12.700</td>
<td>14.300</td>
</tr>
<tr>
<td>6</td>
<td>Friesland en Groningen</td>
<td>230.0</td>
<td>42</td>
<td>4.940.000</td>
<td>1.100.000</td>
</tr>
<tr>
<td>7</td>
<td>Noordzeepolder</td>
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<td>14</td>
<td>50.100</td>
<td>60.200</td>
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<tr>
<td>8</td>
<td>Flevoland</td>
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<td>11</td>
<td>97.400</td>
<td>244.600</td>
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<tr>
<td>9</td>
<td>Veerebroek</td>
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<td>27</td>
<td>58.200</td>
<td>88.600</td>
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<tr>
<td>10</td>
<td>Mastenbroek</td>
<td>47.5</td>
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<td>9.560</td>
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<tr>
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<td>IJsselbeek</td>
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<tr>
<td>12</td>
<td>Wieringen</td>
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<td>22.500</td>
<td>20.800</td>
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<tr>
<td>13</td>
<td>Noord-Holland</td>
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<td>146</td>
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<tr>
<td>14</td>
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<td>Marken</td>
<td>8.6</td>
<td>2</td>
<td>240</td>
<td>2.050</td>
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<td>16</td>
<td>Zuid-Holland</td>
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<td>17</td>
<td>224.200</td>
<td>3.591.000</td>
</tr>
<tr>
<td>17</td>
<td>Leopker- en Krimpenerwaard</td>
<td>48.0</td>
<td>26</td>
<td>31.400</td>
<td>201.500</td>
</tr>
<tr>
<td>18</td>
<td>Alkmaarwaard en Visserslui</td>
<td>86.2</td>
<td>24</td>
<td>39.200</td>
<td>212.800</td>
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<td>19</td>
<td>Lissemonnikoog</td>
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<td>Rozenburg</td>
<td>8.1</td>
<td>7</td>
<td>300</td>
<td>14.000</td>
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<tr>
<td>22</td>
<td>Voerse-Putten</td>
<td>71.0</td>
<td>22</td>
<td>19.500</td>
<td>155.400</td>
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<tr>
<td>23</td>
<td>Hoekse Waard</td>
<td>69.4</td>
<td>31</td>
<td>24.500</td>
<td>83.100</td>
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<tr>
<td>24</td>
<td>Eiland van Dordrecht</td>
<td>37.1</td>
<td>18</td>
<td>4.920</td>
<td>104.800</td>
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<tr>
<td>25</td>
<td>Land van Altena</td>
<td>46.3</td>
<td>13</td>
<td>16.300</td>
<td>51.100</td>
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<tr>
<td>26</td>
<td>Goeree-Overflakkee</td>
<td>44.4</td>
<td>7</td>
<td>22.600</td>
<td>46.500</td>
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<td>27</td>
<td>Schouwen-Duiveland</td>
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<td>21.900</td>
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<td>29</td>
<td>Noord-Beveland</td>
<td>25.7</td>
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<td>7.750</td>
<td>6.570</td>
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# Total: 2,657.4 1,206 1,975.131 11,016.694
Figure A.2: Map of dike rings in the Netherlands [34]
Figure A.3: Dike ring 41 "Land van Maas en Waal"
Figure A.4: Dutch system of main waterways
<table>
<thead>
<tr>
<th>Type de voies navigables</th>
<th>Classe de voies navigables</th>
<th>Automoteurs et chalands</th>
<th>Convois poussés</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of inland waterway</td>
<td>Motor vessels and barges</td>
<td>Type of vessel: générales caractéristiques</td>
<td>Type of convoy- Caractéristiques générales</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Dénomination Designation</td>
<td>Longueur Length</td>
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<tr>
<td></td>
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<tr>
<td></td>
<td></td>
<td>I</td>
<td>Péchiche Barge</td>
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<tr>
<td></td>
<td></td>
<td>II</td>
<td>Kast-Camiinois Campine-Barge</td>
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<td></td>
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<td>III</td>
<td>Gustav Koonings</td>
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<td>IV</td>
<td>Johan Welker</td>
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<td>Grand bateaux Rhénans/Large Rhine Vessels</td>
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Figure A.5: Vessel dimensions per CEMT class. (27)
Hydraulic boundary conditions

This chapter explains some of the boundary conditions that have been identified in Chapter 5. The first sections focus on boundary conditions that arise from the primary flood defence system. Subsequently the boundary conditions that are related to the navigation function are explained.

B.1. Water levels

Table B.1 and Table B.2 show characteristics of the water levels on the river Waal at measuring station "Nijmegen haven" which is located approximately 2 kilometers upstream of Sluis Weurt. Based on reports that refer to expert judgements of Rijkswaterstaat, water levels at Weurt are about 20 centimeters lower than at the measuring location [14] [13].

Table B.1: Overview of water levels at Sluis Weurt

<table>
<thead>
<tr>
<th>Discharge Upper-Rhine [m³/s]</th>
<th>Mean undercut [days/year]</th>
<th>Exceedance frequency [1/year]</th>
<th>Water level &quot;Nijmegen haven&quot; [m + NAP]</th>
<th>Water level &quot;Sluis Weurt&quot; [m + NAP]</th>
</tr>
</thead>
<tbody>
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<td>823</td>
<td>4,84</td>
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<td>353,64</td>
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<td>10,54</td>
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</table>
Table B.2: Continuation of Table B.1

<table>
<thead>
<tr>
<th>Discharge Upper-Rhine [m³/s]</th>
<th>Mean undercut [days/year]</th>
<th>Exceedance frequency [1/year]</th>
<th>Water level &quot;Nijmegen haven&quot; [m + NAP]</th>
<th>Water level &quot;Sluis Weurt&quot; [m + NAP]</th>
</tr>
</thead>
<tbody>
<tr>
<td>5675</td>
<td>358,06</td>
<td></td>
<td>11,18</td>
<td>10,98</td>
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<tr>
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<td>1</td>
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<td>11,38</td>
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<tr>
<td>7095</td>
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<td></td>
</tr>
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<td>7960</td>
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<td>12,12</td>
<td></td>
</tr>
<tr>
<td>8950</td>
<td>7</td>
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<td>12,46</td>
<td></td>
</tr>
<tr>
<td>10085</td>
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</tr>
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<td>11415</td>
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<td>13,21</td>
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</tr>
<tr>
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</tr>
<tr>
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<tr>
<td>15685</td>
<td>984</td>
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<td>14,54</td>
<td></td>
</tr>
<tr>
<td>16000</td>
<td>1250</td>
<td>14,86</td>
<td>14,66</td>
<td></td>
</tr>
</tbody>
</table>

Figure B.1 shows the exceedance frequency for water levels at Sluis Weurt. Difference with the measuring station "Nijmegen haven" is already accounted for.

B.2. Gumbel distribution

The probability density function of the water levels can be approximated by a Gumbel distribution, this is plotted in Figure B.2. The distribution has the following parameters.

\[
\begin{align*}
  f ( h ) &= \alpha \cdot \exp \left[ -\alpha \left( h - u \right) \right] \cdot \exp \left( -\alpha \left( h - u \right) \right) \\
  u &= 6.581 \\
  \alpha &= 0.92
\end{align*}
\]
B.3. Storage capacity

Storage capacity of the Meuse-Waal canal largely determines the acceptable discharge volume over or through the lock complex. Water may flow into the canal both from the river Meuse at Heumen and from the river Waal at Weurt. Probability of simultaneous failure of both lock complexes is assumed to be negligible. Therefore only inflow at Weurt is examined. When the maximum acceptable volume is known the corresponding high water level on the river Waal can be calculated.

An exact calculation of the storage capacity is difficult to make. Therefore only an estimation is done. Conservative assumptions will be made in order to make sure that this estimation is within safe boundaries.

B.3.1. Meuse-Waal canal

The Meuse-Waal canal has an approximate length of 10 kilometer and an average width of 90 meter. This comes down to a surface area of 0.09 square km. Surface area of the harbour at Nijmegen is neglected.

Maximum allowable water level on the canal is NAP + 8.50 meter. The dikes along the canal have a retaining height of NAP + 10.00 meter [1]. Thus a water level difference of 1.50 meter can be stored in the canal. A simple calculation shows that the total volume that can be stored on the canal is equal to:

\[
V_{storage} = 0.09 \text{ [km}^2\text{]} \cdot 1.50 \text{ [m]} = 0.09 \cdot 10^6 \text{ [m}^2\text{]} \cdot 1.50 \text{ [m]} = 135.000 \text{ [m}^3\text{]}
\]

B.3.2. River Waal

Inflow of water is considered for the scenario in which the lock gates are expected to be closed but, for whatever reason, they are not. The model is thus that 'Sluis Weurt' forms an open connection between the river Waal and the Meuse-Waal canal. Discharge into the canal can be calculated using equation B.3 [32]:

\[
Q = 1.9 \cdot B \cdot \Delta h^{3/2}
\]

In which 'B' is the width of the opening (16 meter) and \(\Delta h\) is the water level difference. Discharge of water onto the river Meuse will slightly increase the storage capacity of the Meuse-Waal canal. In the current situation the pumping station at Heumen has a capacity of 6 m³/s. For special circumstances additional capacity of 2.5 m³/s is installed [5]. Additional pumping capacity, however, is neglected since it will take a lot of time to install. Using equation B.3 and the result of B.2 it can be approximated how long it will take for the canal to fill up to NAP + 10.00 meter for different water levels. This is presented in Table B.3.

Figure B.3 shows the general course of high water levels on the river Waal. It shows that a high water wave has a total duration of approximately 400 to 500 hours. The highest water levels are present for several hours,
somewhere between 20 and 30 hours. Combining Figure B.3 with the results in Table B.3 shows that the canal storage capacity is easily exceeded within the duration of a single high water wave. In that case the canal dike system will fail due to overflow and the flood defense system has failed.

Failure of ‘Sluis Weurt’ due to inadmissible inflow thus is assumed to occur when an open connection is combined with the water level on the river Waal is at NAP + 10.00 meter and higher.

Figure B.3: Course of high water levels on river Waal
Probability density functions

This appendix gives the approximation of the probability density functions of the hazards that are considered. Water level is discussed in Section C.1, ship collision is discussed in Section C.2. Both sections start off with a 'basic' distribution which is used to calculate the forces that are exerted on the lock gates. Subsequently, the probability density function of the occurring bending moment in the supporting beams is derived.

C.1. Water level difference

Outer water level

Pressure differences over the lock gates are caused by high water on one side, and low water on the other side. Only high water levels on the river Waal are considered within this research. The water level on the river is considered as the 'outer water level'. The probability density function of the outer water level is known from collected data. It can be approximated by the Gumbel distribution in Figure C.1. The distribution can be written as:

\[
f_h(h) = \alpha \cdot \exp \left[ -\alpha (h - u) - \exp (-\alpha (h - u)) \right]
\]

\[
u = 6.581
\]

\[
\alpha = 0.92
\]

This research only considers water level differences caused by high water levels on the river Waal. These are water levels \( h \geq 7.50 \text{ m + NAP} \). The maximum water level difference occurs at \( h = 15.25 \text{ m + NAP} \). Larger water levels result in overtopping and/or overflow and will not further increase the water level difference.

Figure C.1: Gumbel distribution of the water level on the river Waal [m + NAP]
Inner water level

The 'inner water level' is the water level in the lock chamber. Due to the different lockage procedures, the inner water level partly depends on the outer water level. When a certain outer water level is reached, the inner water level is increased. Hence, the water level difference is discontinuous. This is illustrated in Figure C.2.

Note: only 'positive' bending moments are considered. These occur for high water levels on the river Waal. In reality, there are also 'negative' bending moments that occur during low water levels on the river Waal.

In addition, the maximum considered outer water level is 15.25 m + NAP. In practice, the lock gate will behave as a weir. This means that higher water level differences may occur.

Difference inner and outer water level

In the previous sections, the discontinuous relation between outer water level \( h \) and water level difference \( \Delta h \) has been explained. However, within a lockage procedure, the relation between outer water level \( h \) and water level difference \( \Delta h \) is continuous and linear. Hence, the probability density of the water level difference can be determined for each lockage procedure. The continuous, linear relation is given for each interval in Formula C.2:

\[
\Delta h = \begin{cases} 
0 & h \leq 7.50 \\
7.50 - h & h > 7.50 \\
\frac{h}{2} - 3.75 & 10.00 \leq h < 12.80 \\
\frac{h}{3} - 2.50 & 12.80 \leq h < 15.25 \\
0 & h \geq 15.25 
\end{cases}
\]

From Formula C.2, it can be seen that the relation between \( h \) and \( \Delta h \) is one-one-one for the normal lockage procedure. In the case of the stepped lockage or the stepped retaining procedure, there is a scaled linear
relation. These ‘scaling’ factors are given as:

- Normal lockage: \( \Delta h = h - 7.50 \quad \rightarrow \quad h = \frac{1}{1} \Delta h - 7.50 \)
- Stepped lockage: \( \Delta h = \frac{h}{2} - 3.75 \quad \rightarrow \quad h = \frac{2}{2} \Delta h - 7.50 \)
- Stepped retaining: \( \Delta h = \frac{h}{3} - 2.50 \quad \rightarrow \quad h = \frac{3}{3} \Delta h - 7.50 \)

Formula C.2 can be substituted into Formula C.1 to obtain the probability density of the water level difference \( '\Delta h' \). When substituting, the scaled relation should also be accounted for. Otherwise, the total probability (integral of the probability density) exceeds its initial value. The following distribution is obtained:

\[
(C.3) \quad f_{WD}(\Delta h) = \begin{cases} 
  f_{normal}(\Delta h) & 0.00 \leq \Delta h < 2.50 \\
  f_{steped}(\Delta h) & 1.25 \leq \Delta h < 2.65 \\
  f_{retaining}(\Delta h) & 1.77 \leq \Delta h < 2.58 
\end{cases}
\]

The functions that are mentioned in Formula C.3 directly result from the substitution and are given below:

\[
(C.4) \quad f_{normal}(\Delta h) = \alpha \cdot \exp[-\alpha \cdot ((\Delta h + 7.50) - u)] - \exp(-\alpha \cdot ((\Delta h + 7.50) - u)) \\
 f_{steped}(\Delta h) = 2 \cdot \alpha \cdot \exp[-\alpha \cdot ((2\Delta h + 7.50) - u)] - \exp(-\alpha \cdot ((2\Delta h + 7.50) - u)) \\
 f_{retaining}(\Delta h) = 3 \cdot \alpha \cdot \exp[-\alpha \cdot ((3\Delta h + 7.50) - u)] - \exp(-\alpha \cdot ((3\Delta h + 7.50) - u))
\]

The intervals that are in Formula C.3, have overlap. For clarity, the formula is rewritten into Formula C.5 with separated intervals. The process is also illustrated in Figure C.4.

\[
(C.5) \quad f_{WD}(\Delta h) = \begin{cases} 
  f_{normal}(\Delta h) & 0.00 \leq \Delta h < 1.25 \\
  f_{normal}(\Delta h) + f_{steped}(\Delta h) & 1.25 \leq \Delta h < 1.77 \\
  f_{normal}(\Delta h) + f_{steped}(\Delta h) + f_{retaining}(\Delta h) & 1.77 \leq \Delta h < 2.50 \\
  f_{steped}(\Delta h) + f_{retaining}(\Delta h) & 2.50 \leq \Delta h < 2.58 \\
  f_{steped}(\Delta h) & 2.58 \leq \Delta h < 2.65 
\end{cases}
\]

Finally, the probability density function of the water level difference over the Waal gate is plotted in Figure C.3. Note that the total area under the graph is not equal to 1. This is due to the fact that not all water levels on the river Waal are taken into account. Water levels \( h < 7.50 \) meter will result in a ‘negative’ water level difference over the Waal gate. This means that the inner water level is higher than the outer water level.

![Figure C.3: (Partial) PDF of water level differences over the Waal gate. Based on interval 7.50 < h < 15.25 m + NAP.](image)
Figure C.4: Procedure to find the (partial) PDF of water level differences over the Waal gate. Note that the y-axis has a different scale for each individual lockage procedure.
C.1. Water level difference

The structural model has been introduced in Section 6.2. The water level causes a distributed load on the supporting beams of the lock gate. The distributed load is equal to:

\( q_{wd} = \Delta h \cdot x_{c.t.c.} \cdot \rho_w \cdot g \)

\( \begin{align*} 
\Delta h & = \text{Water level difference [m]} \quad \text{See Figure C.3} \\
x_{c.t.c.} & = \text{Center to center distance beams [m]} \quad \text{Deterministic 2.60 -} \\
\rho_w & = \text{Density of the water [kg \cdot m^{-3}]} \quad \text{Deterministic 1000 -} \\
g & = \text{Gravity [m \cdot s^{-2}]} \quad \text{Deterministic 9.81 -} 
\end{align*} \)

The effective bending moment in the supporting beams that is caused by the distributed load is equal to:

\( M_d = \frac{1}{8} \cdot q_{wd} \cdot l^2 \)

\( \begin{align*} 
q_{wd} & = \text{Distributed load [kNm]} \quad \text{See Formula C.6} \\
l & = \text{Length of the beam [m]} \quad \text{Deterministic 16.70 -} 
\end{align*} \)

The bending moment has a continuous and linear relation with the water level difference. Therefore, it is relatively easy to determine the resulting probability density function of the bending moment. The procedure is similar to the procedure that was used to find the probability density function of the water level difference. Now Formulas C.6 and C.7 are substituted into Formula C.5. The resulting probability density for the bending moment is presented in Figures C.5 and C.6.

![Figure C.5: (Partial) PDF of bending moments in the critical supporting beam. Per lockage procedure.](image)

![Figure C.6: (Partial) PDF of bending moments in the critical supporting beam. All procedures added together.](image)
C.2. Ship collision

C.2.1. Energy distribution of a ship collision

The report "Aanvarrisico’s voor sluisdeuren" [37] gives a lot of information about the energy that is involved in a collision between a ship and a structure. The report uses both a deterministic and a probabilistic method to calculate the collision energy for different velocities of the ship. Even though the Meuse-Waal canal is a CEMT Vb class waterway, a CEMT Va class is used. This is justified by the fact that the CEMT Vb ships generally do not pass through the eastern navigation lock. The situation that has the best match with the case of Sluis Weurt considers a 'Groot Rijnschip' and a water depth of 3.85 meter. See Figure C.7a.

This distribution is based on discrete values. In order to obtain a continuous distribution, an approximation is made. Because of the complex nature of ship collisions and the many variables, a Gaussian distribution is assumed. The approximation is illustrated in Figure C.7b. The mean and standard deviation from Formula C.8 are used, in which the subscript 'E' is used to denote the ship collision energy.

\[
\mu_E = 6000 \text{ [kNm]} = 6 \text{ [MNm]}
\]
\[
\sigma_E = 3000 \text{ [kNm]} = 3 \text{ [MNm]}
\]

C.2.2. Force exerted by the ship

According to the Eurocodes [25], a ship collision can be modeled as a concentrated load. The value of this load can either be taken from a table with indicative values, or it can be calculated. The table with indicative values gives a value per CEMT class. The indicative value for a 'Groot Rijnschip' (CEMT Va) is given as:

\[
F_{\text{indicative}} = 8000 \text{ [kN]} = 8 \text{ [MN]}
\]

The provided formula to calculate the load is:

\[
F_{\text{ship}} = 5.0 \cdot \sqrt{1 + 0.128 \cdot \frac{E}{2}} \text{ [MN]}
\]

Now, the normal approximation of the collision energy can be used to calculate the mean value and standard deviation of the collision force. These are found to be:

\[
\mu_F = \mu_E = 5.0 \cdot \sqrt{1 + 1.028 \cdot \frac{6}{2}} \\
\approx 6.65 \text{ [MN]}
\]
\( \sigma_F = \frac{\partial F(\mu_E)}{\partial E} \cdot \sigma_E \)

\( \sigma_F = \frac{5.0 \cdot 0.128}{2 \cdot \sqrt{6}} \cdot 3 \)

\( \sigma_F \approx 0.72 \) [MN]

The resulting probability density distribution of the force is illustrated in Figure C.8. With regard to the indicative value from C.9, the distribution that has been found seems acceptable. The indicative value is used for situations in which no further calculations are made, therefore it should be a conservative value. Within the distribution the value of 8 [MN] is at the 97th percentile. Hence, the indicative value indeed is a conservative value.

![PDF and CDF of the force caused by ship collision.](image)

**C.2.3. Probability density function of the resulting bending moment**

Based on the structural model that is introduced in Section 6.2, the maximum bending moment in the supporting beams of the lock gates can be calculated. The bending moment is equal to:

\( M_{Ed} = 0.25 \cdot F_{max,m} \cdot l \)

\( M_{Ed} = 0.25 \cdot [3.3 \cdot \sqrt{E} + 5.6] \cdot 16.70 \)

In this equation the collision energy ‘E’ has a probability density function that has the parameters as presented in Section C.2.1. In order to obtain the probability density function of the bending moment \( M_{Ed} \) we need to determine it’s mean value and standard deviation. Assuming a deterministic value for all variables other than the energy, the following values are found:

\( \mu_M = M(\mu_E, l) \)

\( \mu_M = 0.25 \cdot 6.65 \cdot 16.70 \)

\( \mu_M \approx 27.76 [\text{MNm}] \)

\( \sigma_M = \frac{\partial M(\mu_E, l)}{\partial F} \cdot \sigma_F \)

\( \sigma_M = 0.25 \cdot 16.70 \cdot 0.72 \)

\( \sigma_M \approx 3.01 [\text{MNm}] \)
The resulting probability density function of the bending moment due to a ship collision can now be written according to Formula C.16. The probability distribution is illustrated in Figure C.9.

\[
 f_{SC}(M) = \frac{1}{\sqrt{2\pi} \sigma} \exp\left( -\frac{(M - \mu)^2}{2\sigma^2} \right)
\]

Figure C.9: PDF and CDF of the bending moment caused by ship collision.
D.1. Physical decomposition

The physical decomposition of the eastern lock is extracted from the RAMS analysis [12]. The results are summarised in Table 5.2. A first selection of critical elements can be made based on the functions that they contribute to.

Table D.1: Damage states: physical and functional decomposition [12].

<table>
<thead>
<tr>
<th>Element / subsystem</th>
<th>Water retaining</th>
<th>Navigation</th>
<th>Structural element</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Driving system culvert slides</td>
<td>x</td>
<td>x</td>
<td>-</td>
</tr>
<tr>
<td>2 Driving system rolling gates</td>
<td>x</td>
<td>x</td>
<td>-</td>
</tr>
<tr>
<td>3 Lightning protection system</td>
<td>x</td>
<td>x</td>
<td>-</td>
</tr>
<tr>
<td>4 Operating and control system</td>
<td>x</td>
<td>x</td>
<td>-</td>
</tr>
<tr>
<td>5 Operator building</td>
<td></td>
<td>x</td>
<td>-</td>
</tr>
<tr>
<td>6 Bed protection</td>
<td>x</td>
<td>x</td>
<td>-</td>
</tr>
<tr>
<td>7 Exterior lights</td>
<td></td>
<td></td>
<td>-</td>
</tr>
<tr>
<td>8 CCTV system</td>
<td></td>
<td>x</td>
<td>-</td>
</tr>
<tr>
<td>9 Communication system</td>
<td></td>
<td>x</td>
<td>-</td>
</tr>
<tr>
<td>10 Operator building facilities</td>
<td>x</td>
<td>x</td>
<td>-</td>
</tr>
<tr>
<td>11 Soil retaining structures</td>
<td></td>
<td>x</td>
<td>yes</td>
</tr>
<tr>
<td>12 Locking chamber</td>
<td>x</td>
<td>x</td>
<td>yes</td>
</tr>
<tr>
<td>13 Low voltage installations</td>
<td>x</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td>14 Level measuring system</td>
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<td></td>
<td>-</td>
</tr>
<tr>
<td>15 Culvert pumps</td>
<td>x</td>
<td>x</td>
<td>-</td>
</tr>
<tr>
<td>16 Emergency power supply</td>
<td></td>
<td>x</td>
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</tr>
<tr>
<td>17 Maintenance facility</td>
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<td></td>
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</tr>
<tr>
<td>18 Radar system</td>
<td>x</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td>19 Fenders / guiding structures</td>
<td>x</td>
<td></td>
<td>yes</td>
</tr>
<tr>
<td>20 Shipping signals system</td>
<td>x</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td>21 Culvert gates</td>
<td>x</td>
<td>x</td>
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</tr>
<tr>
<td>22 Lock gates</td>
<td>x</td>
<td>x</td>
<td>yes</td>
</tr>
<tr>
<td>23 Lock head (concrete)</td>
<td>x</td>
<td>x</td>
<td>yes</td>
</tr>
<tr>
<td>24 Locking complex terrain</td>
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<td></td>
<td>-</td>
</tr>
<tr>
<td>25 Shallow foundation</td>
<td>x</td>
<td>x</td>
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</tr>
<tr>
<td>26 Sheet piles</td>
<td>x</td>
<td>x</td>
<td>yes</td>
</tr>
<tr>
<td>27 Central control system</td>
<td>x</td>
<td>x</td>
<td>-</td>
</tr>
<tr>
<td>28 Bascule bridges</td>
<td>x</td>
<td></td>
<td>yes</td>
</tr>
</tbody>
</table>
From Table 5.2 the components 12, 21, 22, 23, 25 and 26 are selected for further analysis. Structural elements that are part of the holding basin (11, 19) do not play a role in flood protection. Moreover they only have a facilitating role in the navigation function and are therefore not considered as critical elements. For example structural failure of fenders and guiding structures does result in delay for navigation however these delays are manageable and they do not result in a complete shutdown of the navigation lock. It is expected that as a result these elements will be filtered out anyway in the FMECA. By excluding them on beforehand superfluous work is prevented.

D.2. Failure Mode Effects and Criticality Analysis (FMECA)

The FMECA is a purely qualitative analysis that is used to determine for which elements additional quantification is required. First a description is provided for the components that have been selected from Table 5.2. This holds a description of how the component contributes to the primary functions and possible failure modes. Subsequently probability of these failure modes is estimated and the possible effects are described. Rating of probability and effects is done on a scale from 1 to 5. In which 1 is low probability/effect and 5 is high probability/effect. Two criticality scores are calculated one by summation of the ratings on probability and effect and one by multiplication of these ratings.

D.2.1. Locking chamber (12)

Focus is on the reinforced concrete wall of the locking chamber which is the main structural element. In front of the concrete wall a masonry wall is present. The wall is a gravity wall with a stepped lay-out. The floor of the lock chamber does contribute to the horizontal stability of the lock however is not considered as a structural element. Main constant loads are the soil pressures on the wall. These are balanced by gravity, water pressures and support force from the floor. Other forces on the wall originate mainly from the navigation functions (mooring, waves).

<table>
<thead>
<tr>
<th>Characteristics</th>
<th>Granite blocks 1000 x 1000 x 400 mm (MTTR = 8640 hours)</th>
<th>Reinforced concrete, masonry (MTTR = 8640 hours)</th>
<th>Steel bollards (MTTR = 4320 hours)</th>
<th>Ladders, balustrades, etc. (MTTR = 168 hours)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FMECA scores</td>
<td>Probability rating 1</td>
<td>Effects rating 4</td>
<td>Criticality rating $\sum = 5$, $\prod = 4$</td>
<td></td>
</tr>
</tbody>
</table>

Partial collapse of the wall of the locking chamber might occur due to a ship collision. Another cause might be a decrease in strength due to deterioration. As a result of partial collapse the navigation lock will be blocked.
Failure Mode Effects and Criticality Analysis (FMECA) for navigation due to repair works. The water retaining function is compromised as well. However there might be sufficient residual strength for the water retaining function.

**Probability rating = 1**  
Low failure probability due to proven strength. No big changes are expected in the loads. However, deterioration remains a threat.

**Effects rating = 4**  
Effects of this failure mode are quite high, especially for navigation due to the large MTTR. Effects for the water retaining function depend largely on the residual strength of the system which is hard to quantify.

### D.2.2. Culvert gates (21)

In total there are 6 culvert gates present in the Eastern navigation lock, 2 at each lock head. Spare culvert gates are available in storage to reduce MTTR. Generally the culvert gates are in closed position to retain water and are only opened for the lockage procedure when the lock gates are closed. Exact layout of the culvert gates is not known. Roughly it is built up out of 3 steel bames that are covered with steel plates.

<table>
<thead>
<tr>
<th>Characteristics</th>
<th>Gates: Steel framework with steel plates (MTTR = 4320 hours)</th>
<th>Seal: Wooden frame (MTTR = 720 hours)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>FMECA scores</strong></td>
<td>Probability rating 2</td>
<td>Effects rating 3</td>
</tr>
</tbody>
</table>

**Failure mode(s)**  
Failure can occur either in the beams of the framework or in the steel plates. Possible cause of failure is a large water level difference over the gate in combination with deterioration and/or fatigue. As a result the culvert gate cannot be used anymore for the lockage procedure and will not be able to retain water.

**Probability rating = 2**  
Low failure probability due to proven strength. No big changes are expected in the loads. However, deterioration remains a threat. In addition fatigue increases the failure probability.

**Effects rating = 3**  
Effects are quite high because failure directly results in delay for navigation. The desired additional safety with respect to flooding is lost as well, however the opening has a limited surface area and therefore discharge will be small.
D.2.3. Lock gates (22)

Focus is on the structural elements of the lock gates. These are the elements that retain the water and transfer the loads to the lock heads. The main system is a framework of vertical and horizontal steel beams that is covered with steel plates.

**Failure mode(s)**

Failure of structural elements might occur due to a combination of high water levels and reduced strength. Reduced strength can be caused by deterioration or unidentified damage from earlier exposures. As a result the stress capacity of the steel is exceeded which results in failure of the considered element. Both the navigation function and the water retaining function are then disrupted. Failure might also occur due to failure of the rolling system.

**Probability rating = 3**

Probability of failure is relatively high due to the fact that moving elements of the lock gate are included. The wheels of the carrying trolley endure large forces that vary, especially during closure and opening procedure, which makes them weak spots.

**Effects rating = 4**

Effects of failure are quite high because failure directly results in delay for navigation. The desired safety with respect to flooding is lost as well. Failure of the lock gate results in a large surface area and therefore discharge will be large as well. This causes that loads on the residual parts are likely to be high as well.

<table>
<thead>
<tr>
<th>Characteristics</th>
<th>Gates</th>
<th>L x B x D = 16.7 x 1.01 x 5.2 meter</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical beams</td>
<td>NP12  and NP16 profiles (MTTR = 8640 hours)</td>
<td></td>
</tr>
<tr>
<td>Horizontal beams</td>
<td>Trussed beam, exact layout unknown (MTTR = 8640 hours)</td>
<td></td>
</tr>
<tr>
<td>Air boxes</td>
<td>Exact layout unknown (MTTR = 8640 hours)</td>
<td></td>
</tr>
<tr>
<td>Seal</td>
<td>Wooden frame (MTTR = 720 hours)</td>
<td></td>
</tr>
<tr>
<td>Rolling system</td>
<td>Rail, trolleys (MTTR = 720 hours)</td>
<td></td>
</tr>
<tr>
<td>Other</td>
<td>Gangway, ladders, etc. (MTTR = 168 hours)</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>FMECA scores</th>
<th>Probability rating</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Effects rating</td>
<td>4</td>
</tr>
<tr>
<td>Criticality rating</td>
<td>$\sum$ = 7, $\prod$ = 12</td>
<td></td>
</tr>
</tbody>
</table>

Figure D.3: FMECA: Lock gates

D.2.4. Lock heads (23)

The lock heads directly retain water, provide mooring facilities (for maintenance) and provide space for the lock gates and associated mechanical and electrotechnical facilities. They are constructed with reinforced concrete and in front of the outer walls there is a masonry wall. The lock gates roll over the rail that is founded on the floor that has monolite connection to the lock head at both sides.
Failure mode(s)
Main threat to the lock heads is deterioration of the materials in combination with high water levels. Other loads such as ship collision are prevented by additional structures that create a distance such as the fenders / guiding structures.

Probability rating = 1
Due to the limited number of loads that are likely to occur, also the probability of failure is low.

Effects rating = 4
The elements of the lock heads are essential for good functioning of the navigation lock, therefore high effects of failure can be expected. In general the MTTR of elements is high.

<table>
<thead>
<tr>
<th>Characteristics</th>
<th>Wall</th>
<th>Floor</th>
<th>Basements</th>
<th>Bollards</th>
<th>Seal</th>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Reinforced concrete, masonry (MTTR = 8640 hours)</td>
<td>Monolite concrete (MTTR = 8640 hours)</td>
<td>Concrete (MTTR = 8640 hours)</td>
<td>Steel bollards (MTTR = 4320 hours)</td>
<td>Wooden frame (MTTR = 720 hours)</td>
<td>Ladders, plateau, cover tiles (MTTR = 168 hours)</td>
</tr>
</tbody>
</table>

FMECA scores

| Probability rating | 1  |
| Effects rating     | 4  |
| Criticality rating | \( \Sigma = 5, \Pi = 4 \) |

D.2.5. Shallow foundation (25)

By shallow foundation it is meant that the structure directly rests on the soil, there are no foundation piles present. The foundation makes sure that all external loads and dead weight are transferred to the subsoil.

<table>
<thead>
<tr>
<th>Characteristics</th>
<th>Shallow foundation</th>
<th>Concrete slab (MTTR = 8640 hours)</th>
</tr>
</thead>
</table>

FMECA scores

| Probability rating | 1  |
| Effects rating     | 5  |
| Criticality rating | \( \Sigma = 6, \Pi = 5 \) |
Failure mode(s)
Failure might occur due to extreme water level differences that cause piping. Piping might result in washing out of the subsoil which results in unequally spread pressures and thus undesirable displacements and forces in the structure. Another probability is that the jacking force of the tiles in the locking chamber is lost and results in horizontal instability.

Probability rating = 1
Probability is low due to proven strength. Probability of piping is limited due to the application of sheet piles (26) around the structure.

Effects rating = 5
Failure of the foundation is disastrous for the structure and results in the need to fully replace the structure.

D.2.6. Sheet piles (26)

Two goals are achieved by installment of the sheet piles: stability and prevention of piping. Sheet piles to achieve stability are located at the outer ends of the structure: one at the transition to the canal and one at the transition to the river. They make sure that the tile floor of the lock remains in place and prevent large joints between the tiles.

Other sheet piles are installed all around the navigation lock and prevent flow of water through the surrounding soil bodies. They are also present at each lock head since water level differences occur at these locations and the floor of the chamber is not water tight.

<table>
<thead>
<tr>
<th>Characteristics</th>
<th>Sheet piles stability at NAP - 0.40 meter (MTTR = 8640 hours)</th>
<th>Sheet piles piping at NAP - 3.50 meter (MTTR = 8640 hours)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FMECA scores</td>
<td>Probability rating 1</td>
<td>Effects rating 4</td>
</tr>
<tr>
<td></td>
<td>Criticality rating $\sum = 5$, $\prod = 4$</td>
<td></td>
</tr>
</tbody>
</table>

Figure D.6: FMECA: Sheet piles. Orange lines indicate sheet piles for piping. Light blue lines indicate sheet piles for stability.
Failure mode(s)
Failure might occur due to corrosion of the steel sheet piles, reducing the length over which the water particles have to travel. It might also be the case that water level differences are larger than expected. As a result the design seepage length is too small.

Probability rating = 1
Probability of failure is low due to proven strength although deterioration / corrosion remains a threat.

Effects rating = 4
Effects of failure are quite high since piping might result in loss of overall stability of the structure. That would require total replacement of the structure.

D.2.7. FMECA results

Results of the failure mode effects and criticality analysis are summarised in Table 5.3. Following from the table it is concluded that the lock gates are the most critical elements of the lock complex. This is explained by the fact that they have a relatively large contribution to both functions of the navigation lock which results in large consequences of failure. Also the failure probability is large because the gates are exposed to a wide range of possible hazards and include moving elements that are susceptible to failure.

<table>
<thead>
<tr>
<th>#</th>
<th>Component</th>
<th>Failure probability</th>
<th>Failure effects</th>
<th>Criticality score</th>
<th>Σ</th>
<th>Π</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Lock gates</td>
<td>3</td>
<td>4</td>
<td></td>
<td>7</td>
<td>12</td>
</tr>
<tr>
<td>2</td>
<td>Shallow foundation</td>
<td>1</td>
<td>5</td>
<td></td>
<td>6</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Culvert gates</td>
<td>2</td>
<td>3</td>
<td></td>
<td>5</td>
<td>6</td>
</tr>
<tr>
<td>4</td>
<td>Locking chamber</td>
<td>1</td>
<td>4</td>
<td></td>
<td>5</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Lock heads</td>
<td>1</td>
<td>4</td>
<td></td>
<td>5</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Sheet piles</td>
<td>1</td>
<td>4</td>
<td></td>
<td>5</td>
<td>4</td>
</tr>
</tbody>
</table>

From the FMECA it is concluded that the lock gates are the most critical structural elements of the lock complex. Considering the navigation function it is of no importance which of the lock gate fails; the consequences are the same for each lock gate. The flood defense function on the other hand is dominated by the lock gate in the upper head: the Waal gate. For water levels above NAP + 12.80 meter this lock gate is the only gate with a water retaining function. Hence, the Waal gate is considered as the most critical structural element. For this reason the research focuses on robustness indices that correspond to an initial damage state in which the Waal gate is lost.
System failure probabilities

This appendix presents the underlying calculations of the system failure probabilities that have been presented in Chapter 7.

E.1. General description

In this example calculation the original situation of Sluis Weurt with 3 lock gates is considered. Focus of the calculations is on the eastern navigation lock. Based on the FMECA, only risks related to the lock gates are included for the robustness quantification. A top view of the lock complex is given in Figure E.1. System failure probabilities are considered for a value of $\beta_{\text{design}} = 3.50$.

First, the flood defense function is considered. In Sections E.2.2 and E.2.5 a fault tree of the flood defense function is presented for both the undamaged and the damaged system. The system failure probability is calculated for the initial event of damage due to water level differences in Section E.2.5. Subsequently, in Section E.2.6 the system failure probability is calculated for the initial event of damage due to ship collision. This distinction in initial events allows for a comparison of the robustness with respect to different hazards.

Secondly, the navigation function is considered. The fault trees of this system are presented in Sections E.3.2 and E.3.3. Sections E.3.4 and E.3.5 elaborate on the probabilities of the underlying (series of) events.
E.2. Flood defense system

E.2.1. Relevant lockage procedures

The dike system that lays behind the lock complex has a retaining height of 10.00 m + NAP. Therefore, only water levels on the river Waal above 10.00 m + NAP are considered in the calculation of the flood probability. Hence, only two lockage procedures have to be considered in the fault tree analysis: stepped lockage and stepped retaining. For water levels on the river Waal above 15.25 m + NAP, overflow occurs. Failure of the lock gates is no longer relevant in that situation.

![Figure E.2: Relevant lockage procedures for flooding](image)

E.2.2. Flooding probability of the undamaged system

Only structural failure is considered as a cause of flooding. Structural failure may occur due to water level differences or due to ship collision. In practice, other failure modes may occur as well. In combination with structural failure, these failure modes might result in flooding. An example of such a failure mode is non-closure. Since the focus is on structural robustness, these failure modes are not considered in the calculations.

In the case of the stepped lockage procedure, structural failure of all three lock gates is required in order to cause flooding. This procedure is active for water levels 10.00 < h < 12.80 m + NAP.

![Figure E.3: Fault tree of the undamaged system for top event 'Flooding'](image)
In the case of stepped retaining, only the Waal gate has to fail in order to cause flooding. This is due to the fact that only the Waal gate is equipped with an extension piece. Once the Waal gate fails, the other lock gates do not have sufficient retaining height and they will fail due to overflow. In addition, the probability of structural failure of a lock gate due to ship collision is neglected for this procedure. This is justified by the idea that navigation is obstructed during this procedure. Of course, in practice, there always is the probability that a wayward ship collides with the lock gates.

E.2.3. Flooding probability of the damaged system

The fault tree from Figure E.3 represents the flooding probability for the system without damage. For the robustness calculation, we are interested in the flooding probability of the system with damage. This term is part of the indirect risk, as was introduced in Chapter 4:

\[
P(\text{flood}) = P(\text{hazard}) \cdot P(\text{damage} | \text{hazard}) \cdot P(\text{flood} | \text{damage}) \cdot C(\text{flood})
\]

A complicating factor is the dependence between failure events. These dependencies are especially important for the stepped lockage procedure. The failure events are structural failure of respectively the Waal gate, the middle gate and the canal gate. Dependencies are accounted for by using correlation coefficients. These have been introduced in Chapter 7.

Damage is modelled as failure of the Waal gate. Failure is either caused by water level difference or by a ship collision. Because of dependencies, it is important that a distinction is made in the initial load that led to failure of the Waal gate. Therefore the system failure probability will be calculated for damage caused by water level differences and for damage caused by a ship collision.

Due to failure of the Waal gate, the fault tree from Figure E.3 reduces to a more simplified fault tree. The Waal gate can be removed from the fault tree, as is done in Figure E.4.

![Figure E.4: Fault tree of the damaged system for top event 'Flooding']
E.2.4. Fault tree for hand calculation

In the fault tree of Figure E.4, events with mutual dependencies are located in different gates. This is inconvenient for hand calculations. Therefore the fault tree is rearranged, such that events with mutual dependencies are in the same gate. The newly constructed fault tree is shown in Figure E.5.

Figure E.5: Fault tree of the damaged system for top event 'Flooding'.
E.2.5. Flooding probability, cause of damage ‘water level difference’

Flooding probabilities for the stepped retaining procedure are solely determined the probability of failure of the Waal gate. This probability is known from Chapter 6. Below, the calculations are presented for the probability of flooding for the stepped lockage procedure. This will be done based on the rearranged fault tree in Figure E.6. Each box is elaborated separately. For calculation of the probabilities the following equations are used:

\[
P(\text{’event’} | \text{Waal gate fails due to WD}) = \frac{P(\text{’event’} \cap \text{Waal gate fails due to WD})}{P(\text{Waal gate fails due to WD})}
\]

(E.2)

\[
P(\text{Lock gate fails due to WD}) = 1.56 \times 10^{-04}
\]

\[
\beta_{1\text{-gate,WD}} = -\Phi^{-1}(1.56 \times 10^{-04}) = 3.605
\]

Figure E.6, box 1

Basically, this box comes down to failure of all three lock gates due water level difference. The reliability of a single lock gate is given by Formula E.3. The probability that the three lock gates all fail can be calculated using the approach by Grigoriu & Turkstra (section 7.2.1). The reliability of the parallel system of 3 lock gates is equal to:

\[
\beta_{3\text{-gates,WD}} = 3.605 \cdot \sqrt{\frac{3}{1 + 0.9 \cdot (3 - 1)}} \approx 3.731
\]

The corresponding failure probability of the parallel system of 3 lock gates is equal to:

\[
P(\text{Both gates fail due to WD} \cap \text{Waal gate fails due to WD}) = \Phi(-3.731) \approx 9.52 \times 10^{-5}
\]

Formula E.2 is now used to calculate the probability of the event that both the middle gate and the canal gate fail due to water difference, given the fact that the Waal gate has failed due to water difference:

\[
P(\text{Both gates fail due to WD} | \text{Waal gate fails due to WD}) = \frac{9.52 \times 10^{-5}}{1.56 \times 10^{-04}} \approx 0.610
\]
The reliability of the parallel system of 2 lock gates is equal to:

$$\beta_{2\text{-gates}} = 3.605 \cdot \sqrt{\frac{2}{1 + 0.9 \cdot (2 - 1)}} \approx 3.699$$

The corresponding failure probability of the parallel system of 2 lock gates is equal to:

$$P(\text{Middle gate fails due to WD} \cap \text{Waal gate fails due to WD}) = \Phi(-3.699) \approx 1.08E-04$$

Formula E.2 is now used to calculate the probability of the event that the middle gate fails due to water level difference, given the fact that the Waal gate has failed due to water level difference:

$$P(\text{Middle gate fails due to WD} | \text{Waal gate fails due to WD}) = \frac{1.08E-04}{1.56E-04} \approx 0.695$$

The event of a ship collision on the canal gate is independent from the event of failure due to a water level difference. Therefore, correlations do not have to be taken into account. The failure probability of the canal gate due to ship collision can be taken directly from Chapter 6:

$$P(\text{Canal gate fails due to SC} | \text{Waal gate fails due to WD}) = P(\text{Canal gate fails due to SC}) \approx 4.36E-06$$

Because of their independence, the joint probability of the events in box 2 can be simply written as the product of the individual event probabilities:

$$P(\text{Middle gate fails due WD} \cap \text{Canal gate fails due SC} | \text{Waal gate fails due WD}) = 0.695 \cdot 4.36E-06 \approx 3.03E-06$$

In principal, box 2 and box 3 have the same occurrence probability. The only difference is the sequence of their events. Therefore, the joint probability of the events in box 3 is also equal to:

$$P(\text{Middle gate fails due WD} \cap \text{Canal gate fails due SC} | \text{Waal} \cap 10.00 < h < 12.80) = 0.695 \cdot 4.36E-06 \approx 3.03E-06$$

The reliability of a single lock gate, with respect to ship collision, can be calculated as:

$$P(\text{Lock gate fails due to SC}) = 4.36E-06$$

$$\beta_{1\text{-gate,SC}} = -\Phi^{-1}(4.36E-06) \approx 4.447$$

The damaged system can be interpreted as a parallel system of 2 lock gates with a reliability equal to:

$$\beta_{2\text{-gates,SC}} = 4.447 \cdot \sqrt{\frac{2}{1 + (-0.5) \cdot (2 - 1)}} \approx 8.893$$

Because of independency, the joint probability of the two events in box 4 can be written as:

$$P(\text{Both gates fail due to SC} | \text{Waal gate fails due to WD}) = \Phi(-8.893) \approx 2.96E-19$$
E.2. Flood defense system

Table E.1: Flooding probabilities of the damaged system for $\beta_{\text{design}} = 3.50$. Initial hazard is water level difference.

<table>
<thead>
<tr>
<th>Water level</th>
<th>$P(H)$</th>
<th>$P(D \mid H)$</th>
<th>$P(F \mid D)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>10.00 - 12.80</td>
<td>3.89E-02</td>
<td>4.02E-03</td>
<td>6.10E-01</td>
</tr>
<tr>
<td>12.80 - 15.25</td>
<td>2.93E-03</td>
<td>1.39E-03</td>
<td>1.00E+00</td>
</tr>
<tr>
<td>&gt;15.25</td>
<td>3.44E-04</td>
<td>1.00E+00</td>
<td>1.00E+00</td>
</tr>
</tbody>
</table>

Summary of the results: flooding probability due to water level difference

The calculated probabilities can now be entered into the fault tree of Figure E.6, the resulting fault tree is given in Figure E.7. Table E.1 gives an overview of all probabilities that are required for robustness quantification.

Figure E.7: Fault tree gate for the stepped lockage procedure, including the calculated probabilities for initial hazard ‘water level differences’ and $\beta_{\text{design}} = 3.50$
E.2.6. Flooding probability, cause of damage 'ship collision'

This section considers the situation in which the damage state is caused by ship collision. Thus, the hypothetical scenario is checked in which the Waal gate has collapsed due to a ship collision. The failure probability of the damaged system is calculated by determining the values in the fault tree of figure E.6. For calculation of the probabilities the following equations are used:

\[(E.4) \quad P(\text{event} | \text{Waal gate fails due to SC}) = \frac{P(\text{event} \cap \text{Waal gate fails due to SC})}{P(\text{Waal gate fails due to SC})}\]

\[(E.5) \quad \beta_{1\text{-gate,SC}} = -\Phi^{-1}(4.36E-06) = 4.447\]

Figure E.6, box 1

The reliability of a single lock gate, with respect to water level difference is equal to \(\beta = 3.605\) (given by Formula E.3). The damaged system can be interpreted as a parallel system of 2 lock gates with a reliability equal to:

\[\beta_{2\text{-gates,WD}} = 3.605 \cdot \sqrt{\frac{2}{1 + 0.9 \cdot (2 - 1)}} \approx 3.699\]

Now, the joint probability of the two events in box 1 can be written as:

\[P(\text{Both gates fail due to WD | Waal gate fails due to SC}) = \Phi(-3.699) \approx 1.08E-04\]

Figure E.6, box 2

The reliability of the parallel system of 2 lock gates is equal to:

\[\beta_{2\text{-gates,SC}} = 4.447 \cdot \sqrt{\frac{2}{1 + (-0.5) \cdot (2 - 1)}} \approx 8.893\]

The corresponding failure probability of the parallel system of 2 lock gates is equal to:

\[P(\text{Canal gate fails due to SC \cap Waal gate fails due to SC}) = \Phi(-8.893) \approx 2.96E-19\]

Formula E.4 is now used to calculate the probability of the event that the canal gate fails due to ship collision, given the fact that the Waal gate has failed due to ship collision:

\[P(\text{Canal gate fails due to SC | Waal gate fails due to SC}) = 2.96E-19 / 4.36E-06 \approx 6.79E-14\]

The event of water level difference on the middle gate is independent from the event of failure of the other gates due to a ship collision. Therefore, correlations do not have to be taken into account. The failure probability of the middle gate due to water level difference can be taken directly from Chapter 6:

\[P(\text{Middle gate fails due to WD | Waal gate fails due to SC}) = P(\text{Middle gate fails due to WD}) \approx 1.56E-04\]

Because of their independence, the joint probability of the events in box 2 can be simply written as the product of the individual event probabilities:

\[P(\text{Middle gate fails due WD \cap Canal gate fails due SC | Waal gate fails due SC}) = 1.56E-04 \cdot 6.79E-14 \approx 1.06E-17\]
Figure E.6, box 3

In principal, box 2 and box 3 have the same occurrence probability. The only difference is the sequence of their events. Therefore, the joint probability of the events in box 3 is also equal to:

\[ P(\text{Canal gate fails due } WD \cap \text{Middle gate fails due } SC \mid \text{Waal gate fails due } SC) = 1.56E-04 \cdot 6.79E-14 \approx 1.06E-17 \]

Figure E.6, box 4

Basically, this box comes down to failure of all three lock gates due to ship collision. The reliability of a single lock gate is given by Formula E.5. The probability that the three lock gates fail can be calculated using the approach by Grigoriu & Turkstra (section 7.2.1). The reliability of the parallel system of 3 lock gates is:

\[ \beta_{3\text{-gates,SC}} = 4.447 \cdot \sqrt{\frac{3}{1 + (-0.5) \cdot (3 - 1)}} \approx \infty \]

The corresponding failure probability of the parallel system of 3 lock gates is equal to:

\[ P(\text{Both gates fail due to SC } \cap \text{Waal gate fails due to SC}) = \Phi(-\infty) \approx 0.00 \]

Using Formula E.4 it is found that:

\[ P(\text{Both gates fail due to SC } \mid \text{Waal gate fails due to SC}) = \frac{0.00}{4.36E-06} \approx 0.00 \]

Summary of the results: flooding probability due to ship collision

The calculated probabilities can now be entered into the fault tree of Figure E.6, the resulting fault tree is given in Figure E.8. Table E.2 gives an overview of the probabilities that are required for robustness quantification.

Table E.2: Flooding probabilities of the damaged system for \( \beta_{\text{design}} = 3.50 \). Initial hazard is water level difference.

<table>
<thead>
<tr>
<th>Water level</th>
<th>( P(H) )</th>
<th>( P(D \mid H) )</th>
<th>( P(F \mid D) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>10.00 - 12.80</td>
<td>4.36E-06</td>
<td>1.00E+00</td>
<td>1.08E-04</td>
</tr>
<tr>
<td>12.80 - 15.25</td>
<td>0.00E-00</td>
<td>1.00E+00</td>
<td>1.00E+00</td>
</tr>
<tr>
<td>&gt;15.25</td>
<td>0.00E-00</td>
<td>1.00E+00</td>
<td>1.00E+00</td>
</tr>
</tbody>
</table>

Figure E.8: Fault tree gate for the stepped lockage procedure, including the calculated probabilities for initial hazard 'ship collision' and \( \beta_{\text{design}} = 3.50 \).
E.3. Navigation system

E.3.1. Relevant lockage procedures

For the analysis of delay probabilities due to structural failure, the normal lockage procedure and the stepped lockage procedure are relevant. For other cases, there already is a delay due to natural boundary conditions.

For both lockage procedures, it can be stated that a short delay occurs when only the Waal gate fails. When a second lock gate fails, a long delay occurs.

When only one lock gate fails, this causes a short delay. The other two gates are still available and can be used for the lockage procedure. Nonetheless, it is required to float the middle gate into the position of the Waal gate. This causes a short delay. In addition, it is no longer possible to follow the stepped lockage procedure for water levels above 10.00 m + NAP. These water levels normally don't last for more than a few days and therefore do not cause long delays.

When two lock gates fail, this causes a long delay. All lockage procedures require at least 2 operational lock gates. Repair of the damaged gates or construction of new lock gates requires time spans in the order of weeks or months. The presence of the western navigation lock, however, strongly reduces the delay for navigation.

The distinction in short and long delays has also been made in Chapter 5 which discusses the consequences delay for navigation. In the model that is used, duration of the delay doesn't depend on the water level that is present.

E.3.2. Delay probability of the undamaged system

In the undamaged situation, the navigation function fails when ships are delayed. This happens frequently because of low water levels on the river Waal. However, this research focuses on the structural robustness of the lock complex and therefore interest goes out to delays that are caused by structural failure. Figure E.9 shows a fault tree with the series of events that result in delay of navigation.

![Fault Tree](image)

Figure E.9: Fault trees of the undamaged system for top event 'delay'. The fault tree for long delay contains a voting gate. The value ‘2’ indicates that at least two of the underlying events have to occur to pass the gate.
E.3.3. Delay probability of the damaged system

Now, the damaged system is considered. Again, damage is modelled as loss of the Waal gate. Based on the fault trees from Figure E.9, the probability of short delay for the damaged system is equal to 1. The probability of a long delay for the damaged system, in which the Waal gate has failed, is given by the fault tree from Figure E.10.

Structural failure of the Waal gate results in a short delay in any case. Although lockage might still be possible with the remaining two gates, procedures require that there is always a gate present in the Waal head. To achieve this, the middle gate is floated into the position of the Waal head. This operation requires some time and navigation is delayed for some days. Therefore the probability of a short delay for the damaged system is equal to 1.

Once the middle gate has been floated into its new position in the Waal head, the possibility of a long delay is still present. In case one of the two remaining lock gates fails as well, lockage is no longer possible. Repair of the damaged gates or construction of new lock gates is required. This covers time spans of weeks or maybe even months. Hence, navigation is delayed for a long period. Note: even though the middle gate is floated into the Waal head, it is still referred to as the middle gate.

Again, dependencies have to be accounted for. Therefore, the fault tree is slightly adjusted such that the dependent events are again in the same gate. The approach is similar to the approach that was used for the flood defense function.

Figure E.10: Fault tree of the damaged system for top event 'long delay'. The fault tree for short delay is not presented because failure of the Waal gate always results in a short delay: \( P(\text{short delay} | \text{damage}) = 1. \)
E.3.4. Long delay probability, cause of damage 'water level difference'

This section considers the situation in which the damage state is caused by water level difference. Thus, the hypothetical scenario is checked in which the Waal gate has collapsed due to a large water level difference. The failure probability of the damaged system is calculated by determining the values in the fault tree of figure E.10. For calculation of the probabilities the following equations are used:

(E.6) \[
P ( \text{`event' | Waal gate fails due to WD} ) = \frac{P ( \text{`event' } \cap \text{Waal gate fails due to WD})}{P ( \text{Waal gate fails due to WD})}
\]

(E.7) \[
P ( \text{Lock gate fails due to WD}) = 1.56E-04
\]

\[
\beta_{1\text{-gate,WD}} = -0.1^{-1}(1.56E-04) = 3.605
\]

**Figure E.10, box 1**

The probability that either of the two lock gates fails can be written as:

\[
P ( A_1 \cup A_2 | A_3 ) = P ( A_1 | A_3 ) + P ( A_2 | A_3 ) - P ( A_1 \cap A_2 | A_3 )
\]

\[
A_1 = \text{Middle gate fails due to WD}
\]

\[
A_2 = \text{Canal gate fails due to WD}
\]

\[
A_3 = \text{Waal gate fails due to WD}
\]

The probability that a 2nd lock gate fails due to water level difference can be calculated using Grigoriu & Turkstra. The reliability of the parallel system of 2 lock gates is equal to:

\[
\beta_{2\text{-gates,WD}} = 3.605 \cdot \sqrt{\frac{2}{1 + 0.9 \cdot (2 - 1)}} \approx 3.699
\]

\[
P ( A_1 \cap A_3 ) = \Phi(-3.699) \approx 1.08E-04
\]

Formula E.6 is now used to calculate the probability of the event that the canal gate fails due to water level difference, given the fact that the Waal gate has failed due to water level difference:

\[
P ( A_1 | A_3 ) = 1.08E-04 / 1.56E-04 \approx 0.695
\]

The probability that the middle gate fails is equal to the probability that the canal gate fails:

\[
P ( A_2 | A_3 ) = P(A_1| A_3) \approx 0.695
\]

Now the joint failure probability of the lock gates has to be calculated.

\[
\beta_{3\text{-gates,WD}} = 3.605 \cdot \sqrt{\frac{3}{1 + 0.9 \cdot (3 - 1)}} \approx 3.731
\]

The corresponding failure probability of the parallel system of 3 lock gates is equal to:

\[
P ( A_1 \cap A_2 \cap A_3 ) = \Phi(-3.731) \approx 9.52E-05
\]

\[
P ( A_1 \cap A_2 | A_3 ) = 9.52E-05 / 1.56E-04 \approx 0.610
\]

Finally the probability of the events in box 1 is equal to:

\[
P ( A_1 \cup A_2 | A_3 ) = P ( A_1 | A_3 ) + P ( A_2 | A_3 ) - P ( A_1 \cap A_2 | A_3 )
\]

\[
P ( A_1 \cup A_2 | A_3 ) = 0.695 + 0.695 - 0.610 = 7.99E-01
\]
The reliability of a single lock gate, with respect to ship collision, can be calculated as:

\[ P(\text{Lock gate fails due to SC}) = 4.36E-06 \]

\[ \beta_{1\text{-gate,SC}} = -\Phi^{-1}(4.36E-06) = 4.447 \]

The probability of the gate can be written as:

\[ P(B_1 \cup B_2 | A_3) = P(B_1 | A_3) + P(B_2 | A_3) - P(B_1 \cap B_2 | A_3) \]

\( B_1 = \text{Middle gate fails due to SC} \)

\( B_2 = \text{Canal gate fails due to SC} \)

\( A_3 = \text{Waal gate fails due to WD} \)

The damaged system can be interpreted as a parallel system of 2 lock gates with a reliability equal to:

\[ \beta_{2\text{-gates,SC}} = 4.447 \cdot \sqrt{\frac{2}{1 + (-0.5) \cdot (2 - 1)}} \approx 8.893 \]

\[ P(B_1 \cap B_2 | A_3) = \Phi(-8.893) \approx 2.96E-19 \]

The resulting probability of box 2 is:

\[ P(B_1 \cup B_2 | A_3) = P(B_1 | A_3) + P(B_2 | A_3) - P(B_1 \cap B_2 | A_3) \]

\[ P(B_1 \cup B_2 | A_3) = 4.36E-06 + 4.36E-06 - 2.96E-19 \approx 8.72E-06 \]

**Summary of the results: delay probability due to water level difference**

The calculated probabilities can now be entered into the fault tree of Figure E.10, the resulting fault tree is given in Figure E.11. Table E.3 gives an overview of the probabilities that are required for robustness quantification. These values are based on Figure E.11 and the input from Chapter 5 and Chapter 6.

<table>
<thead>
<tr>
<th>Water level</th>
<th>( P(H) )</th>
<th>( P(D \mid H) )</th>
<th>( P(F \mid D) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Short delay</td>
<td>3.49E-01</td>
<td>6.67E-04</td>
<td>1.00E+00</td>
</tr>
<tr>
<td>Long delay</td>
<td>3.49E-01</td>
<td>6.67E-04</td>
<td>7.90E-01</td>
</tr>
</tbody>
</table>

Table E.3: Delay probabilities of the damaged system for \( \beta_{\text{design}} = 3.50 \). Initial hazard is water level difference.
E.3.5. Long delay probability, cause of damage 'ship collision'

This section considers the situation in which damage is caused by ship collision. Thus, the hypothetical scenario is checked in which the Waal gate has collapsed due to a ship collision. The failure probability of the damaged system is calculated by determining the values in the fault tree of figure E.10.

Figure E.10, box 1

The probability that either of the two lock gates fails can be written as:

\[
P(A_1 \cup A_2 | B_3) = P(A_1 | B_3) + P(A_2 | B_3) - P(A_1 \cap A_2 | B_3)
\]

\[
A_1 = \text{Middle gate fails due to WD} \quad A_2 = \text{Canal gate fails due to WD} \quad B_3 = \text{Waal gate fails due to SC}
\]

The reliability of a single lock gate, with respect to water difference, is known from Chapter 6:

\[
P(\text{Lock gate fails due to WD}) = 1.56E-04
\]

\[
\beta_{1\text{-gate,WD}} = -\Phi^{-1}(1.56E-04) = 3.605
\]

The probability that both lock gates fail due to water level difference corresponds to the reliability:

\[
\beta_{2\text{-gates,WD}} = 3.605 \cdot \sqrt{\frac{2}{1 + 0.9 \cdot (2 - 1)}} \approx 3.695
\]

\[
P(A_1 \cap A_2 | B_3) = \Phi(-3.695) \approx 1.08E-04
\]

The resulting probability of box 1 is:

\[
P(A_1 \cup A_2 | B_3) = P(A_1 | B_3) + P(A_2 | B_3) - P(A_1 \cap A_2 | B_3)
\]

\[
P(A_1 \cup A_2 | B_3) = 1.56E-04 + 1.56E-04 - 1.08E-04 = 2.04E-04
\]

Figure E.10, box 2

The probability that either of the two lock gates fails can be written as:

\[
P(B_1 \cup B_2 | B_3) = P(B_1 | B_3) + P(B_2 | B_3) - P(B_1 \cap B_2 | B_3)
\]

\[
B_1 = \text{Middle gate fails due to SC} \quad B_2 = \text{Canal gate fails due to SC} \quad B_3 = \text{Waal gate fails due to SC}
\]

The probability that a 2nd lock gate fails due to ship collision can be calculated using Grigoriu & Turkstra. See also box 2 in Section E.3.4. The reliability of a parallel system of 2 lock gates is equal to:

\[
\beta_{2\text{-gates,SC}} = 4.447 \cdot \sqrt{\frac{2}{1 + (-0.5) \cdot (2 - 1)}} \approx 8.893
\]

\[
P(B_1 \cap B_3) = \Phi(-8.893) \approx 2.96E-19
\]

The probability of the event that the canal gate fails due to ship collision, given the fact that the Waal gate has failed due to ship collision, is:

\[
P(B_1 | B_3) = 2.96E-19 / 4.36E-06 \approx 6.79E-14
\]
The probability that the middle gate fails is equal to the probability that the canal gate fails:

$$P(B_2 | B_3) = P(B_1 | B_3) \approx 6.79 \times 10^{-14}$$

Now the joint failure probability of the lock gates has to be calculated.

$$\beta_{3\text{-gates,SC}} = 4.447 \sqrt{\frac{3}{1 + (-0.5) \cdot (3 - 1)}} \approx \infty$$

The corresponding failure probability of the parallel system of 3 lock gates is equal to:

$$P(B_1 \cap B_2 \cap B_3) = \Phi(-\infty) \approx 0.00$$

Finally the probability of the events in box 1 is equal to:

$$P(B_1 \cup B_2 | B_3) = P(B_1 | B_3) + P(B_2 | B_3) - P(B_1 \cap B_2 | B_3)$$

$P(B_1 \cup B_2 | B_3) = 6.79 \times 10^{-14} + 6.79 \times 10^{-14} - 0.00 = 1.36 \times 10^{-13}$

**Summary of the results: delay probability due to ship collision**

The calculated probabilities can now be entered into the fault tree of Figure E.10, the resulting fault tree is given in Figure E.12. Table E.4 gives an overview of the probabilities that are required for robustness quantification. These values are based on Figure E.11 and the input from Chapter 5 and Chapter 6.

| Delay        | $P(H)$     | $P(D | H)$ | $P(F | D)$ |
|--------------|------------|-----------|-----------|
| Short delay  | 4.36E-06   | 1.00E+00  | 1.00E+00  |
| Long delay   | 4.36E-06   | 1.00E+00  | 2.04E-04  |

Figure E.12: Original situation: delay probabilities of the damaged system due to ship collision.
Direct and indirect risks
<table>
<thead>
<tr>
<th>Range</th>
<th>Direct risk (total)</th>
<th>Flood risk (total)</th>
<th>Direct risk (per initial hazard)</th>
<th>Flood risk (per initial hazard)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.50 - 10.00</td>
<td>€152,014</td>
<td>€2,903,275</td>
<td>€16,335,852</td>
<td>€16,335,852</td>
</tr>
<tr>
<td>10.00 - 12.80</td>
<td>€4,000,000</td>
<td>€3,673,264</td>
<td>€84,468</td>
<td>€84,468</td>
</tr>
<tr>
<td>Direct risk (water diff.)</td>
<td>€0.00</td>
<td>€0.00</td>
<td>€0.00</td>
<td>€0.00</td>
</tr>
<tr>
<td>Direct risk (ship coll.)</td>
<td>€0.00</td>
<td>€0.00</td>
<td>€0.00</td>
<td>€0.00</td>
</tr>
</tbody>
</table>

Note: The tables and diagrams represent various risk assessments and calculations related to flood risk and direct risk, including indirect risks and robustness requirements.
Both direct risk and indirect risk are approximately zero. Division by zero results in an error.