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Influence and applicability of the Eurocode

For locks that are part of the Flood Defense System

Master of Science Thesis

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Preface

This graduation report is part of my master program Hydraulic Structures, faculty of Civil Engineering and Geosciences at Delft University of Technology. The work consist a research about the influence and applicability of the Eurocode on the design of locks that are part of the Flood Defense System.

In the department of Hydraulic Engineering, working with the Eurocode led to many questions, in particular concerning the interaction between the Eurocode and the Leidraad Kunstwerken (a guideline based on the Water Law). Because of my internship at Grontmij before graduating, I encountered this subject. Grontmij raised the issue as a graduation topic, and it is a privilege to offer this engineering firm up-to-date information, allowing Grontmij to offer more cost-effective designs to their clients.

I would like to thank the members of my thesis committee for the effort they put into this report, *prof. dr. ir. S.N. Jonkman* for his critical comments on my thesis and *prof. ir. A.C.W.M. Vrouwenvelder* for his advice about the Eurocode and the Leidraad Kunstwerken. I would especially like to thank *Ir. W.F. Molenaar* for his support during my studies, for all his feedback, for assisting me with the research process and for helping me to structure my thinking now and then. Thanks to all the colleagues at Grontmij for their professional support and hours of consultation, especially *Ing. D.H.M. Adriaansens* for his assistance during my time at Grontmij and his critical comments. *Ir. J. De Vlieger* was also of great help with some technical issues in my thesis.

Summary

Motivation

Since the entrance of the Eurocode a lot of commotion emerged for the hydraulic engineer. The entrance of the Eurocode entailed a problem for the hydraulic structures that are part of the flood defense system. According the law the locks that are part of the flood defense system should meet the requirements based on the most critical standard. Previously for hydraulic structures the critical standard was Leidraad Kunstwerken (based on the Waterwet). This standard was more conservative than the NEN (based on the Woningwet). Now the NEN is replaced with the Eurocode, a higher safety class is added that is more conservative as the LK. So, hydraulic structures are forced by the law to be designed based on the Eurocode, while the Eurocode is actually made for buildings and bridges. So, the question follows: would it be correct to apply the Eurocode for designing locks that are part of the flood defense system.

Conflict

This research started by studying both standards and search where they are in conflict with each other. The Eurocode prescribes requirements regarding the structural design of a structure. Apparently, in the structural framework, the LK does only prescribe partial factors for hydraulic loads (of which the partial factor for water level differences is the most important). This is directly where the main problem of this research emerges; the partial factor for water level differences. This is the level where Eurocode and LK crossing paths. There are more hydraulic loads prescribed in the LK, except these loads are not really influential. In all the other components (in the framework of strength and stability) no conflicts emerge.

The conflict regarding the partial factor for water level differences arises among others from the applied reliability index. The LK and Eurocode prescribe a different reliability index concerning the strength and stability for locks that are part of the flood defense system. This resulted in the discussion of which reliability should be used. The law prescribes the Eurocode, but the LK is based on a more corresponding theoretical framework. This discussion is illustrated as follow:



Figure 0-1: The discussion which was created since the Eurocode became valid.

There are of course more elements which are affected by the Eurocode. Those are the result from the change of NEN to the Eurocode (for example the partial load factors for own weights). These elements are not specifically related with the hydraulic structures, so the LK is not involved for those elements. In this research this standard is just dealing with the water level differences.

Further Approach

After the preliminary study the objectives of the research can be defined more specifically. A distinction is made in the influence and the applicability of the Eurocode. Concerning the influence, the practical (structural and costs) are research. Regarding the applicability, the partial factor for water level differences is studied in more detail. See below for the specific research questions of each case study:

Influence of the Eurocode (**Case 1**)

What are the additional costs for a lock design in case the Eurocode is applied instead of the NEN/LK, and in which extent do the structural elements influence that?

Applicability of the Eurocode

In what extent is the current determination method (according LK) for the partial factor for water level differences and those applied parameter, applicable and how should the method be adopted to make it more accurate?

a. Is the influence coefficient which is prescribed by the ISO and Eurocode used correctly in the determination method (**Case 2**)

b. Are the reliability classes represented in the Eurocode applicable in the determination method for water level differences? This question asks for more research than this report can offer. For this reason the question is limited and rewritten as: are the failure systems between hydraulic structures and buildings comparable? Therefore, is the investment (from LK to EC) lower as the decrease in risk? (**Case 3**)

Findings

Case 1

The magnitude of the additional cost caused by the Eurocode is not even clear yet. This case study gives an indication of how this matter should be approached by comparing a lock design made in Eurocode and NEN/LK. A comparison of the cost estimations reveals a small increase in costs, approximately 0.9% of the total costs. The case study discusses the design elements separately. A large portion of the additional cost comes from the lock head and gate. In particular, the partial factor for water level differences plays an important role.

Case 2

The determination method described in the LK utilizes a semi-probabilistic method, including several parameters. One of these parameters, the influence coefficient, may be used incorrectly. The currently applied influence coefficient is prescribed by the ISO standards (which are also not based on hydraulic structures). By applying the wrong influence coefficient in the semi-probabilistic method, a deflection could arise in the reliability index (between the semi-probabilistic method (Method I) and probabilistic method (Method II)). The deflection should be calculated for the partial factors for the loads and for the strength. If a deflection is found for the loads in one way, the deflection of the strength will be the other way around. Only by combining these to deflection the final deflection can be calculated, which should be smaller than 0.5.

In this case study, this deflection is only calculated concerning the load part (partial factor for water level differences). In some situations the deflection was slightly larger than 0,5 (approximately 0.6). It seems realistic to assume that this deflection will be restored to 0.5 (or smaller) by the combining the deflection on the strength side. To be sure, the deflection of the strength should be calculated as well. This requires more

research. But basically, it seems that the influence coefficient is used in the correct way for the partial factor for water level differences.

Case 3

The third case study focuses on the applicability of the reliability index prescribed by the Eurocode. This reliability is based on buildings and bridges, so it does not seem applicable to be used in a flood defense system. A flood defense system has a much larger failure tree than buildings have. The system includes different failure mechanisms, such as overflow and closing of flood doors.

In this last case study an analysis is made of the effect on the risk by applying the Eurocode for locks instead of applying the LK. The change in standard results in a lower failure probability of the top event, which subsequently leads to a decrease in risk. An increased required investment balances this decrease. To make the investment useful, the increased investment should be lower than the decrease in risk. This analysis includes some fault tree analyses. Finally the existing dike-rings are assessed with the composed model. From an economic point of view, it seems that the dike-rings with higher exceedance frequencies (1/1250, 1/2000, 1/4000) do not benefit from the use of the Eurocode. This is in contrast to the lower exceedance frequency (1/10000).

So, the use of the Eurocode results in a certain decrease of the risk. This improvement can also be achieved by improving other failure mechanisms. To stick to the structures, if for example the failure mechanisms overflow will be improved, it is found that an increase of the gate with 3 cm results in the same improvement in case the gate is designed in with the Eurocode. This seems to be a less expensive measure.

(Main) Recommendations

- There are strong arguments for designing the structural elements (of a lock that is part of the flood defense system) separately. Elements such as the gates, stability of the lock head and protection against piping should be designed in the highest category (reliability class 3). The other elements should be designed in reliability class 2. With this recommendation, unnecessary costs are avoided

- It would be recommended to start research a more accurate influence coefficient for the differences in water level. The advice would be to rewrite it as a function of the water level differences. It results in a more accurate approximation for the partial factor for water level differences, provided that the detailed method is used.

- In addition on the case study from chapter 8, an actual dike ring could be applied to analyze. Up till now, the case study applies only not existing dike systems. It just showed the relations and importance of the elements involved. With a concrete example, the consequence can be defined specifically as well as the applied correlations in the dike ring and the investments. If such an analysis is carried out for several dike rings and are registered, the results could probably conclude something about the applicability of the Eurocode for the flood defense system.

- It would be recommended to consider alternative measures to improve the dike-ring. For example the mechanism “overflow”: by slightly increasing the height of the gates, the same improvement of the dike-ring is reached. It is worth to consider these alternatives since it can save a lot of money in the design

Symbols and Abbreviations

Symbols

β	= Reliability index
β_N	= Reliability index for a reference period of N years
$\phi(..)$	= Standard normal cumulative function
$\varphi(..)$	= Standard normal density function
P_f	= Failure probability
F_{exd}	= Exceedance frequency
ξ	= Failure space of a certain element in a fault tree
f_N	= Reference period of the structure
t	= Time
i	= Inflation
r	= Interest
I_0	= Value at t = 0
I_t	= Value at t = t
I_{EC}	= Investment of a lock design based on the Eurocode
I_{NEN}	= Investment of a lock design based on the NEN
γ_R	= Strength partial factor
γ_S	= Load partial factor
k_R	= Indication of the characteristic value for the strength
k_S	= Indication of the characteristic value for the load
α_R	= Influence coefficient for the strength
α_S	= Influence coefficient for the loads
V_R	= Variance for the strength
V_S	= Variance for the load
B	= Decimeringshoogte
u	= Mean value (Gumbel) of the outer water level
h_b	= Inner water level
u’	= $u - h_b$
ρ_c	= Correlation
C	= Consequence
PV	= Present value
Z	= Reliability function
R	= Resistance
S	= Loads
Δh	= Water level difference
N	= Number of locks
γ_w	= Load partial factor for water level differences
$\rho_{d,w}$	= Water density
g	= Gravitation
$W_{y,pl}$	= Plastic section modules
$f_{y,sp}$	= Nominal value of the strength of the steel

$f_{y,\mu}$	= Mean value of the strength of the steel
d	= Thickness of the plate (of the gate)
∑H	= Sum of the horizontal forces
∑V	= Sum of the vertical forces
∑M	= Sum of the bending moments
f	= Friction (tan(2/3ϕ))
ϕ	= Friction angle
L_h	= Horizontal creep length
L_v	= Vertical creep length
$C_{w,creep}$	= Creep factor
γ_{pipng}	= Partial factor for piping
X^*	= Design point of X
μ_x	= Mean value of X (Normal distributed)
σ_X	= Standard deviation of X (Normal distributed)
$\mu_{L,X}$	= Mean value of X (Lognormal)
$\sigma_{L,X}$	= Standard deviation of X (Lognormal)
α_{Gumbel}	= Location parameter for Gumbel distribution
β_{Gumbel}	= Scale parameter for Gumbel Distribution
f(…)	= Function
$P_{f,LK}$	= Failure probability in case the LK is applied
$P_{f,EC}$	= Failure probability in case the Eurocode is applied

Abbreviations

	NL	UK
WoW:	Woningwet	Housing Law
WaW:	Waterwet	Water Law
BB:	Bouwbesluit	Building agreement
LK:	Leidraad Kunstwerken	Guideline Water Retaining Structures
EC:	Eurocode	Eurocode
WTI:	Wettelijk Toetsinstrumentarium	Lawful Verification Institute
ROK:	Richtlijnen Ontwerp Kunstwerken	Guidelines Designing Structures
RBK:	Richtlijnen Bestaande Kunstwerken	Guidelines Existing Structures
RWS:	Rijkswaterstaat	Government body
HR:	Hydraulische randvoorwaarden	Hydraulic boundary conditions
VTV:	Voorschrift Toetsing Veiligheid	Prescription Verification Safety
RC:	Betrouwbaarheid klasse	Reliability class
HS:	Waterbouwkundige constructies	Hydraulic Structures
S/S:	Sterkte/Stabiliteit	Strength/Stability
MHW:	Gemiddeld hoogwater	Mean High Water
MLW	Gemiddeld laagwater	Mean Low Water
OLR:	Overeengekomen laagste rivierstand	Agreed lowest river water level
TC:	Totale kosten	Total costs

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1 Introduction

1.1 Background

In effect from the 1st of April 2012, the National Building Agreement (Bouwbesluit) considers the Eurocode as the statutory standard to demonstrate the structural safety and durability of structures. The Eurocode will replace the NEN 6700 series.

For hydraulic engineers, much uncertainty arose with the publication of the Eurocode. Like the NEN, the Eurocode is designed for buildings and bridges. In the period of the NEN, a translation was made for hydraulic structures that are part of a flood defense system. This translation is published in the Leidraad Kunstwerken (LK). LK has a safety approach that connects better with hydraulic structures, and is more conservative than the NEN.

The interesting thing about these standards is that in case of contradictions, the most stringent law should be applied. Since the 1th of April 2012, the standards for buildings and bridges (the Eurocode) became more severe as the standards for hydraulic structures (which are part of the flood defense system (LK)). And because the law prescribes to use the most stringent standard; the Eurocode should be applied for hydraulic structures. This is where the confusion starts.

The fundamentals of hydraulic structures and buildings or bridges are different. The failure systems of these two types of structures not equal. Hydraulic structures which are part of the flood defense system. The main objective of the system is the protection of a certain area against flooding. This is a much more extensive system as for buildings. This implies some doubt about the applicability of the Eurocode (the reliability indexes) for hydraulic structures.

In addition, the Eurocode did not prescribed any partial factor for hydraulic loads. In the period of the NEN, the LK generated a determination method in which the hydraulic load became approachable. If the reliability index of the Eurocode is applied in this method, the partial factor for hydraulic loads increases significantly. This brought the attention to some limitations of the determination method. These will be explained more extensively in the report.

Furthermore, the actual impact of the Eurocode on a design for hydraulic structures of not clear is well. It seems obvious that stricter standards results in higher costs. But does the Eurocode lead to massive changes in a design and which elements are decisive?

1.2 Structure of the Report

This report includes two phases (illustrated in the red and green box in Figure 1-1). The first section (Chapter 2 to 5) describes the relevant theory concerning the Eurocode and a lock design. The theory will be used to convert the general problem (see Section 1.1) to a more specific problem, a problem that is approachable. Chapter 5 summarizes these specific problems. In the second phase (Chapter 6 to 9) three cases are presented. The first case gives an impression of the influence of the Eurocode on the costs of a lock design. The second case analyses the determination of the load partial factor for water level differences. The last case discusses the influence of the Eurocode on the failure system (the flood defense system). Figure 1-1 gives a full overview of the structure of the report.

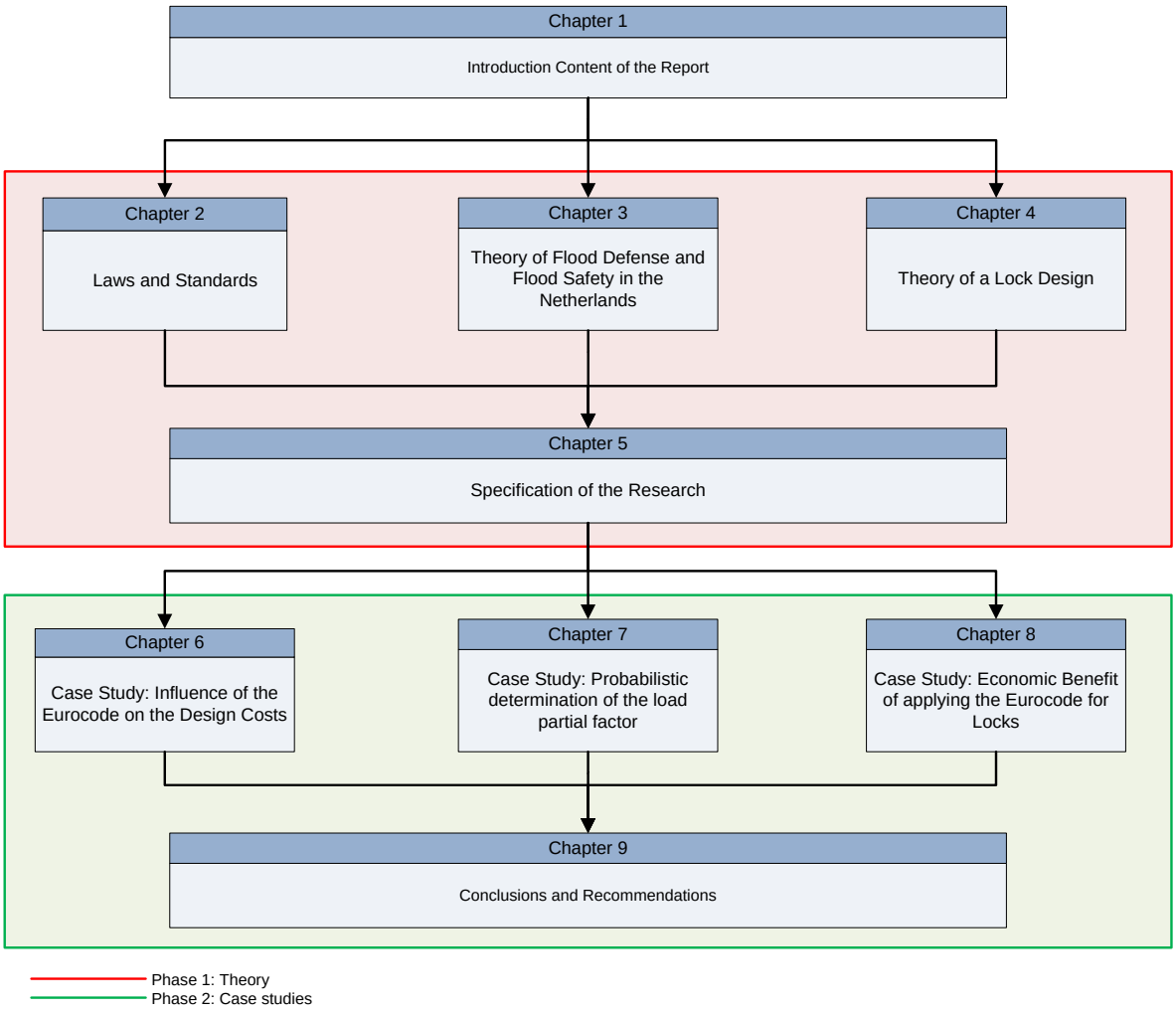
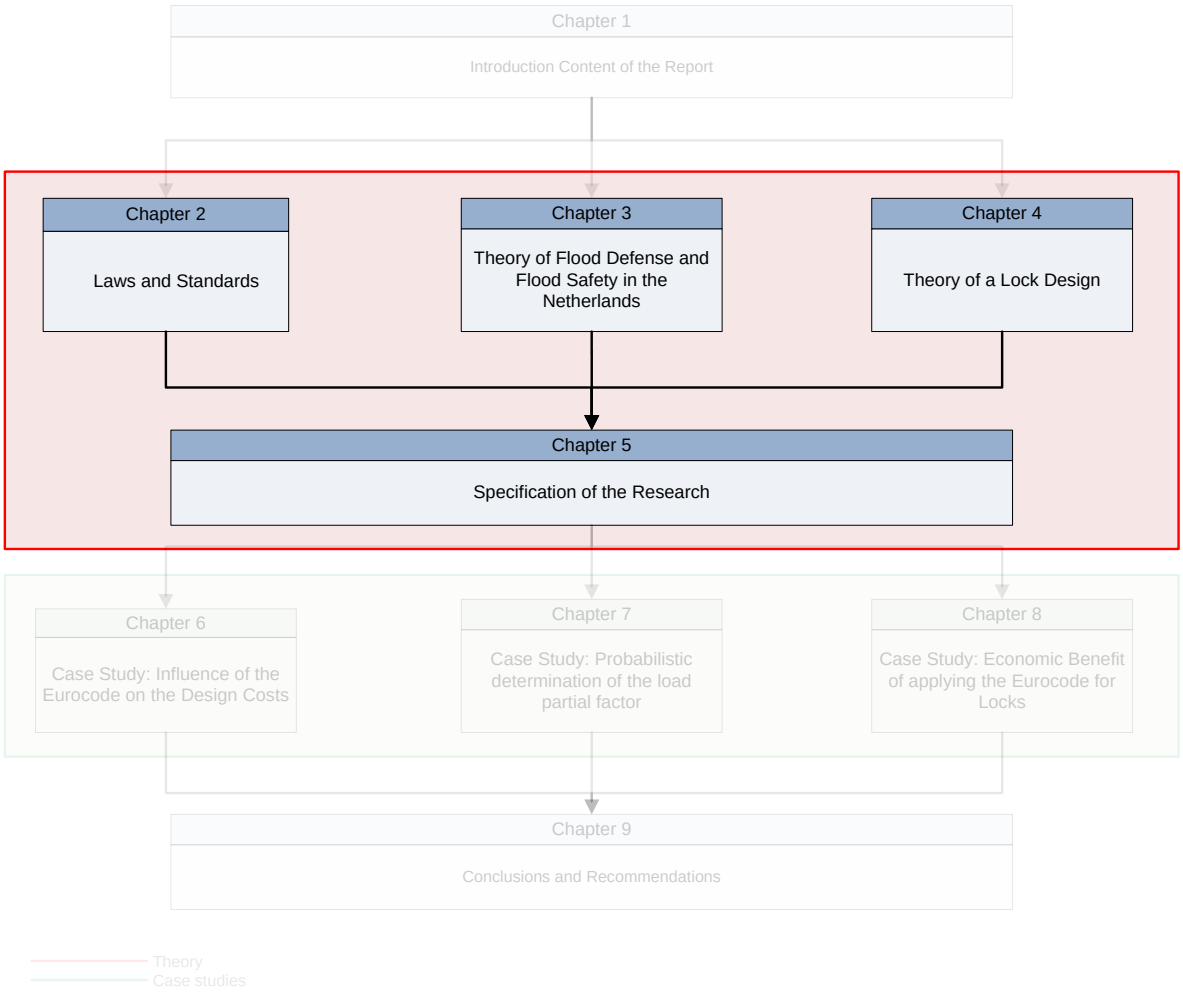


Figure 1-1 Structure of the report

The theory presented in Chapter 3 relates to the case study in Chapter 7. The theory in Chapter 4 links with the case study in Chapter 8. Chapter 6 is a more freestanding case study, which addresses to the technical and financial consequences of using the Eurocode for locks. From the three case studies, conclusions and recommendations can be formed. Chapter 9 lists these conclusions and recommendations.

Phase 1


Chapter 1
Introduction Content of the Report
Chapter 2
Laws and Standards
Chapter 3
Theory of Flood Defense and Flood Safety in the Netherlands
Chapter 4
Theory of a Lock Design
Chapter 5
Specification of the Research
Chapter 6
Case Study: Influence of the Eurocode on the Design Costs
Chapter 7
Case Study: Probabilistic determination of the load partial factor
Chapter 8
Case Study: Economic Benefit of applying the Eurocode for Locks
Chapter 9
Conclusions and Recommendations
Theory
Case studies

2 Laws and Standards

2.1 Relation between Laws, Standards and Guidelines

In the Netherlands, structures have to meet a certain level of safety. Usually national laws declare which standards are valid and which are not. There are guidelines that support the standards. A guideline explains how to approach a certain structural design. Figure 2-1 shows the relationships between the laws, standards and guidelines that are relevant for hydraulic structures in the Netherlands.

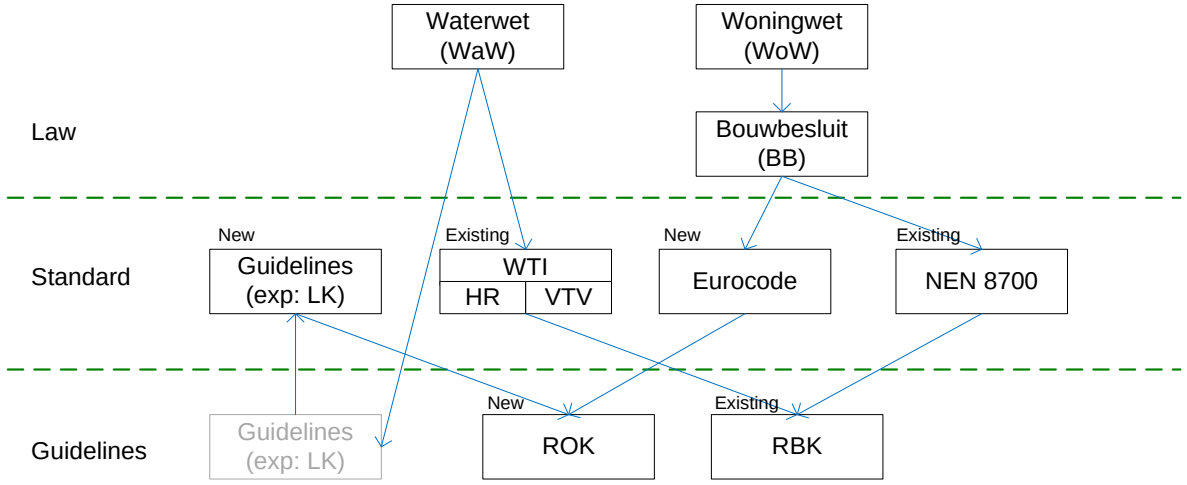


Figure 2-1: Overview of the relationships between law, standards, guidelines and manuals [10]. For translations see the abbreviations in the front matter of this report.

2.1.1 Laws

In the Netherlands, there are two laws that are relevant to the design of hydraulic structures. The most obvious one is the Waterwet (WaW) which has been in effect since 2009. This law combines several older laws into one new law. One of those old laws was the Wet op Waterkering. This law imposes requirements with regard to the safety against flooding. These are now implemented in the Waterwet.

The second law that is applicable for navigation locks is the Woningwet (Bouwbesluit). The Woningwet (WoW) is a law that set requirements for buildings and bridges. In this law, there is no reference to hydraulic structures. Still, a navigation lock should always satisfy the requirements in the WoW. Therefore, the condition for the design or assessment of structures on the highest level should always satisfy both laws (Water and Housing). In case of contradictions, the most stringent law will be applied.

2.1.2 Standards

Leidraad Kunstwerken

The requirements concerning the Waterwet are described in several guidelines, which are prescribed by the government. The most relevant guideline for this report is the Leidraad Kunstwerken (LK) . Strictly speaking, the LK should be seen as a guideline rather than a standard: the LK gives recommendations and not requirements. However, because all water-retaining structures that are part of the flood defense system are funded by the government, the principle, “He who pays the piper calls the tune”, ensures that the recommendation can be seen as requirements. For this reason, the LK is treated as a standard in this thesis.

The LK explains how an engineer should approach a hydraulic structure that is part of the flood defense system, provided it concerns a new design. This guideline provides recommendations for the design and management of a water retaining structure (of the flood defense system), in order to optimize the different functions and values that such a structure can have. Further, the structure should meet the functional and constructive principles, requirements and boundary conditions. In particular, this guide discusses the aspects concerning the water-retaining function of these structures. For the other functions, the guideline refers to relevant NEN-publications (now Eurocode), other guidelines, technical reports, and design documents. Currently the LK is being rewritten in order to generate a better connection with the Eurocode.

Existing hydraulic structures are controlled by the Wettelijk Toetsinstrumentarium (WTI). The Waterwet states that every six (now 12) years the overall conditions of the primary flood defense should be reported. This will result in a national assessment of the safety of the flood defenses. The goal of the assessment is to maintain a clear overview of the safety that our flood defenses offer. The assessment serves as starting point for decisions on the necessary measures for improvement of the primary flood defense.

Eurocode and NEN

The Woningwet prescribed requirements concerning structures in the Building Agreement (Bouwbesluit). Subsequently, for technical requirements the Building Agreement prescribed the Eurocode as a standard. The Eurocode made its official debut on 1 April 2012. Before that time, the NEN-standards were valid. In Table 2-1 the development of the standards over time are presented. Except for the Eurocode, all the other releases where nationally oriented. The Eurocode is a standard that is used in selected countries in Europe.

Table 2-1: Development of the standards (regarding Woningwet) in time

Year	Standard
1950	NEN 1050
1972	NEN 3850 TGB
1990	NEN 6702 TGB
2012	NEN-EN 1990 Eurocode

The Eurocode (as well as the NEN) mainly refers to buildings and bridges. Obviously, it contains many requirements relating to the loads, verifications, materials, durability, reliability, etc.

For existing buildings and bridges, a different approach to the standard is used (NEN 8700). As described in NEN 8700, for economic reasons it is permitted to use reductions in the safety margins. This will prevent a large part of all structures having to be adapted because of some slightly stricter requirements. Today, with the recent transition of the standards (NEN to Eurocode) and their different safety classes, there is a lot of attention to existing structures.

The Eurocode is divided into ten parts: EN 1991 to EN 1999 (see Figure 2-2). EN 1990 establishes principles and requirements for the safety, serviceability and durability of structures, describes the basis for their design and verification and gives guidelines for related aspects of structural reliability. EN 1991 Eurocode 1 provides comprehensive information on all actions that should be considered in the design of buildings and other civil engineering works. EN 1992 to EN 1996 and EN 1999 apply to the design of buildings and other civil engineering works concerning different materials. EN 1997 Eurocode 7 applies to the geotechnical aspects of the design of buildings and other civil engineering works. EN 1998 Eurocode 8 concerns the requirements with regard to seismic design.

The Eurocode contains a National Annex. The Eurocode is the same in every country, but the National Annexes differ from each other. Each country has a certain degree of freedom to determine its own National Annex. The Annex contains, for example, the load and material partial factors. The design approaches for geotechnical structures are described in the National Annex as well.

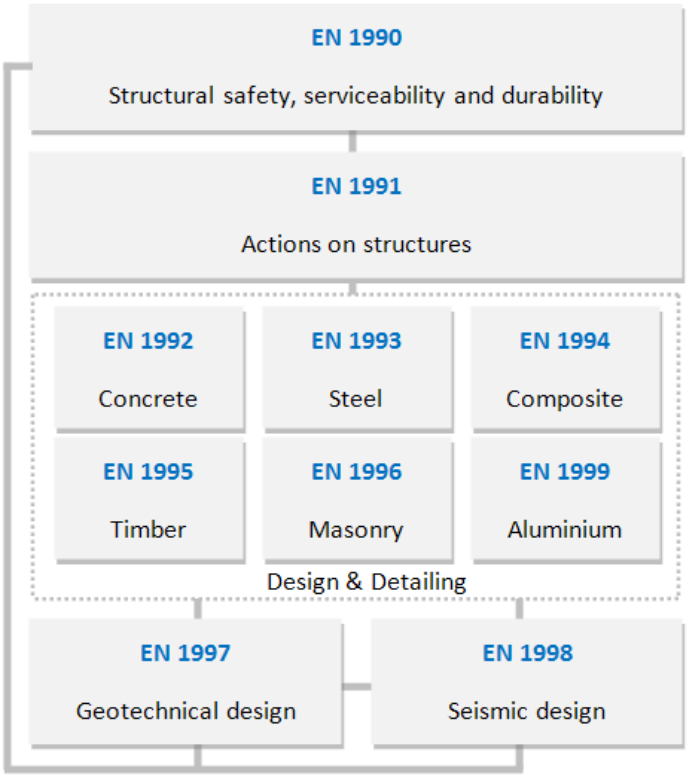


Figure 2-2: Structure of the Eurocode [1]

ISO
ISO (International Organization for Standardization) is an international standard-setting body composed of representatives from various national standards organizations. The organization promotes worldwide proprietary, industrial and commercial standards. In the field of hydraulic engineering the ISO also has its influences. Usually the NEN or the Eurocode refers to certain standards which are published by the ISO. Therefore, the ISO is not a frequently used standard. However, in this research one of the publications of the ISO is under discussion. Later on in this report the concerning discussion will be clarified in more detail.

2.1.3 Manuals

In addition to the standards, there are manuals that give additional requirements to specific structural elements. Three manuals that are frequently used for hydraulic structures are the Richtlijnen Ontwerp Kunstwerken (ROK), Richtlijnen Beoordeling Kunstwerken (RBK) and the guidelines developed by the Commissie voor de Uitvoering van Research (CUR). These manuals were established on instructions of the Rijkswaterstaat (RWS). The requirements for which these manuals give additional recommendations are in fact sufficient. This explains why the manuals should be seen as guidelines and not standards.

ROK
ROK is the manual that is used for new structures. To create a safe and reliable design, ROK sets requirements and boundary conditions. Most of those can also be found in the Eurocode or in the National Annex. In addition to those requirements, the RWS has a couple of specific requirements. The reason for this is that some Eurocode requirements are not strict enough or are not even available in the Eurocode. RWS tried to close this gap in insufficient or missing requirements by creating the ROK, which can be used for designing a structure. For hydraulic structures in particular, the ROK can be useful. Some hydraulic loads, which are not named in the Eurocode, are appointed and explained in the ROK.

RBK
RBK is a manual that is used for existing structures. In the past, existing structures were assessed with the same standards that were used for new structures. In 2004, a change developed in the standards with regard to existing structures. In 2012, this distinction in type of structure was admitted by the law (Bouwbesluit 2012). The RBK was created by RWS, TU Delft and TNO. In the RBK, requirements are set for the safety and usability of existing structures. These guidelines are consistent with the objective of uniform assessment of structural safety.

CUR
For the design of earth-retaining structures, a third standard will be introduced: the CUR (now also replaced by the Eurocode). CUR is a research centre that gives advice and recommendations on particular subjects in construction industry. It contains technical rules that enable the use of new materials and/or new (constructive) applications, for which currently no generally accepted rules exist. CUR-166 (2005) gives recommendations regarding sheet pilling. Further, it gives advice on calculation methods and on the soil parameters. CUR-166 has its own classification of safety with regard to sheet pilling.

2.2 Safety classes in standards

2.2.1 New Structures

It is not necessary to design all structures to the same safety requirements; therefore, different safety classes were introduced. If the consequences are low, a low reliability class is chosen. If the consequences are high, a high reliability class is chosen. The description according the Eurocode:

RC 1: Low consequence for loss of human life, *and* economic social or environmental consequences small or negligible (Examples: agricultural buildings where people do not normally enter, as storage buildings, greenhouses);
RC 2: Medium consequence for loss of human life, and economic, social or environmental consequences are considerable (Example: residential and office buildings);
RC 3: High consequence for loss of human life, *or* economic social or environmental consequences are very great (Example: grandstands and concert hall).

Each class is linked to reliability index β. A high consequence class is linked to a high reliability index. The reliability index stands for the probability of failure ($P_{f,structure}$) of the structure.

$$P_{f,structure} = \phi(-\beta) \tag{2.1}$$

in which β is the required reliability index for the structure and ϕ(…) stands for the standard normal distribution.

Table 2-2 shows an overview of the classes that are applied for the different standards. It is important to add the period of reference to these indices. For these three standards, the period of reference is set to 50 years.

Table 2-2: Classification of CUR, NEN and EC

CUR-166 (6th edition) [5]		NEN-6700 [2]		Eurocode [1]	
Class	β_{50}	Class	β_{50}	Class	β_{50}
RC 1	2,5				
		RC 1	3,2	RC1	3,3
RC 2	3,4	RC 2	3,4		
		RC 3	3,6	RC2	3,8
RC 3	4,2			RC3	4,3

No classification is set for the Waterwet. The reliability requirement this standards asks from structures is variable. The reliability of the structure depends on the exceedance frequencies (which stand for a certain critical water level) that are set for certain area. The exceedance frequency depends on the economic value of the area. In the Netherlands these areas are called dike rings. This subject will be discussed more extensively in Chapter 3. The frequencies can be transformed in a reliability index. In that case the standards in the Woningwet and Waterwet can be compared. The conversion from frequency to reliability index follows from Equation 2.2:

$$\beta = \phi^{-1}(P_{f,N}) = \phi^{-1}(\xi \cdot frequency / f_N) \tag{2.2}$$

in which β is the reliability index of the structure under discussion and stands ϕ(…) for the inverse of the standard normal distribution. $P_{f,N}$ is the probability of failure of the structure for a certain frequency and reference period. The ξ-factor indicates the failure space and is defined in the LK, which will be explained more extensive in Chapter 3. This factor is set to 0.01 for failure due to strength and stability. The f_N value stands for the reference period.

Table 2-3: Classification of LK, NEN and EC

LK [3]			NEN-6700 [2]			EC [1]		
Frequency	β_1	β_{50}		β_1	β_{50}		β_1	β_{50}
1/1250	4,31	3,35	RC1	4,19	3,2	RC1	4,2	3,3
1/2000	4,42	3,48	RC2	4,35	3,4			
1/4000	4,56	3,66	RC3	4,51	3,6	RC2	4,7	3,8
1/10000	4,75	3,89						
						RC3	5,1	4,3

For hydraulic structures that are part of a flood defense system the most critical safety class will be used. It does not matter to which law this safety class is linked. In Figure 2-3 an overview of the comparison of the standards is shown. It can be seen that in the period when the NEN was used, in some cases the NEN was more conservative and in other cases the LK was more conservative. Today, with the Eurocode in force, in most cases the Eurocode is more conservative than the LK. With a reliability of 4.3 of RC3 in the Eurocode and the highest reliability of 3.89 in the LK, a considerable gap exists between the Eurocode and LK.

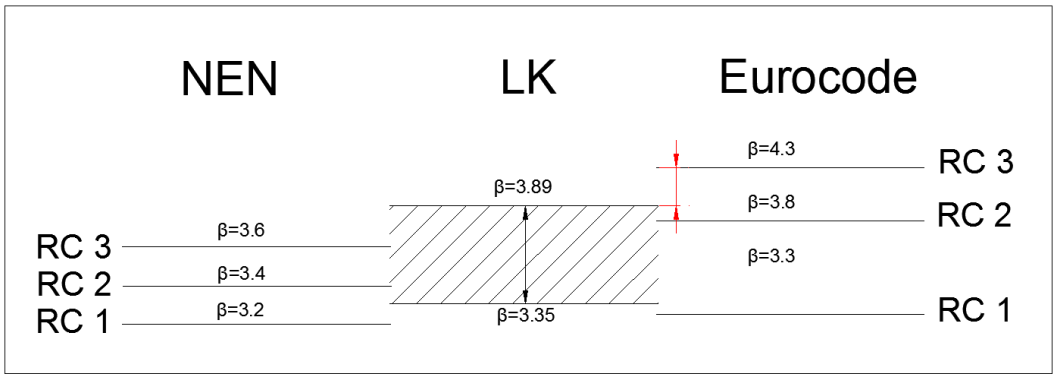


Figure 2-3: Reliability index of the WW, EC and NEN standards

2.2.2 Existing structures

The reliability index for restoration or verifying of existing locks will be established by reducing the reliability index used for new structures.

For restoration: $$\beta_{restoration} = \beta_{new} - 0.5 \quad (2.3)$$

For verification: $$\beta_{verification} = \beta_{new} - 1.5 \quad (2.4)$$

in which $\beta_{restoration}$ and $\beta_{verifying}$ are the critical reliability indexes for restoration and verification. β_{new} is the reliability index for newly built structures. These numbers come from the TNO report "Veiligheidsbeoordeling bestaande bouw" [9].

For existing structures, the reference period is lower than for new structures, it is set on a period of 15 years. Only this does not lead to any reduction of the reliability index. What does cause a reduction is lowering the critical extreme loads. For a lower reference period, a lower maximum load is expected. In addition, the economical optimum reliability (for theoretical explanation see Section 2.4.1) is lower for existing structures. This is illustrated in Figure 2-4. The investment to increase the reliability for an existing structure is much higher than for a new building (where everything is still on paper). The graphic on the left shows the development of the risk for new structures and the one on the right shows the same for existing structures.

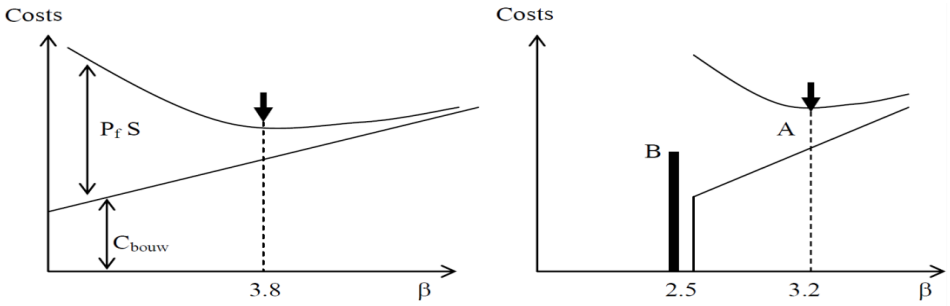


Figure 2-4: Differences in economical optimum reliability for new and existing structures [9] The left figure indicates the total cost for a new design, with Pf as the failure probability, S as the consequence and C as the construction costs. On the right, the total costs are expressed for existing structures, with B as the current risk (in €) and A as the optimum costs in case the reliability increases due to an investment. For this example, it is cheaper to leave the structure as it is because B is lower than A.

Besides the economical optimum, there is always a critical lower boundary based on the criterion of human safety. This lower boundary is prescribed by the Eurocode and depends on the reliability class. Table 2-4 defines the critical reliability for every class. In most cases, for the structures that will be restored or verified, a reference period of 15 years is applied. An overview of the reliability indices that should be applied are shown in Table 2-4.

Table 2-4: Reliability index for restoration and verification [9]

	Lower boundary [9]	Period t	New (EC)	Restoration	Verification
RC1	β > 2.3 -1.10 log t = 1,0	15	3.3	2.8	1.8
RC2	β > 3.4 -0,75 log t = 2,5	15	3.8	3.3 (3,1)	2.5*
RC3	β > 4.0 -0,60 log t = 3,3	15	4.3	3.8 (3,6)	3.3*

* For these values the critical reliability is valid

2.3 Relation Standards and Designing a Structure

The standards discussed so far, all represent a reliability to which a structure should be designed. The next step is to bring the reliability into a design. For the standards, this will be done with a semi-probabilistic approach. In a semi-probabilistic approach, partial factors are used to insert a safety level into the design (see Section 2.4.3). The partial factors are presented in the standards.

The factors are applied to parameters with an uncertain behaviour. In a design, the partial factors will be applied from three perspectives, as there are three types of partial factors:

- Load factors
- Model factors
- Material factors

Partial load factors are used to increase (negative effect on the structure) or decrease (positive effect) the loads. The model factor covers the uncertainty in the resistance model. Finally, the material factors are used to bring the uncertainty of material properties into account. Figure 2-5 gives an overview of the standards and their manners of influence on a design.

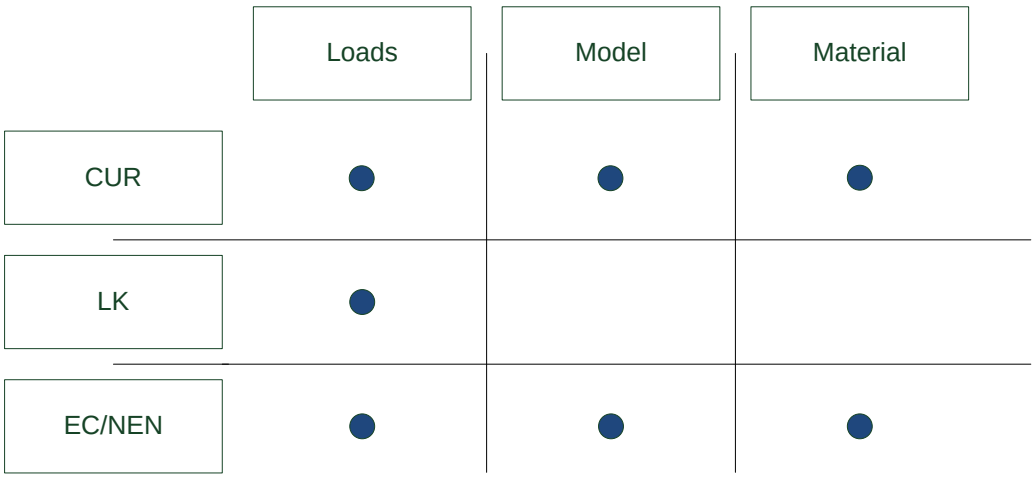


Figure 2-5: Relation standards and applied types of partial factors

Figure 2-5 clearly shows that LK has influence on a design by imposing requirements on just the loads. The LK demands certain reliabilities against flooding. Flooding is a failure mechanism that occurs if water levels are extreme, which explains that the LK only presents partial factors for the loads. It does use a material correction factor, but this will be used on the load as well.

2.4 Fundamentals of Safety in Standards

The fundamentals of defining safety in the standard are based on risk analysis, fault tree analysis and probabilistic engineering techniques, see Figure 2-6. These fundamental calculations and analysis are not published in the standards themselves. A short description of these elements follows in this section [24].

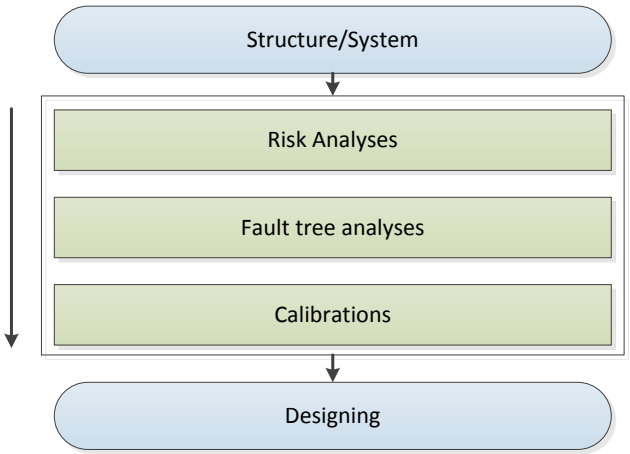


Figure 2-6: Fundamentals of approaching the safety for a structure of a system

2.4.1 Risk analyses

In the process of finding the optimum total costs, risk analyses play an important role, in these the total costs is defined as the sum of the investment costs and the risk, and the risk is defined as the consequence multiplied with the failure probability, see:

$$Risk = P_{failure} \cdot Consequence \tag{2.5}$$

$$Mimimum\ Costs\ = \min(I(Pf) + PV(Pf \cdot C)) \tag{2.6}$$

in which:

I(..)	= The investment in relation with the failure probability
PV	= Present value
Pf	= Failure probability
C	= Consequence

β

The reliability classifications in the Eurocode are ranked by the consequence caused by failure of the structure. These classifications are the result of several analyses. For a given consequence and investment the optimal failure probability of the structure can be found from Equation 2.5. Structures with small consequences will be linked with a higher failure probability (lower reliability index). Structures with high consequences results in a lower optimum failure probability (higher reliability index). This principle is illustrated in Figure 2-7. The risks for the highest and lowest reliability index are sketched (not to scale). In the figure it has been assumed that the investment (as a function of the reliability) for both classes is equal.

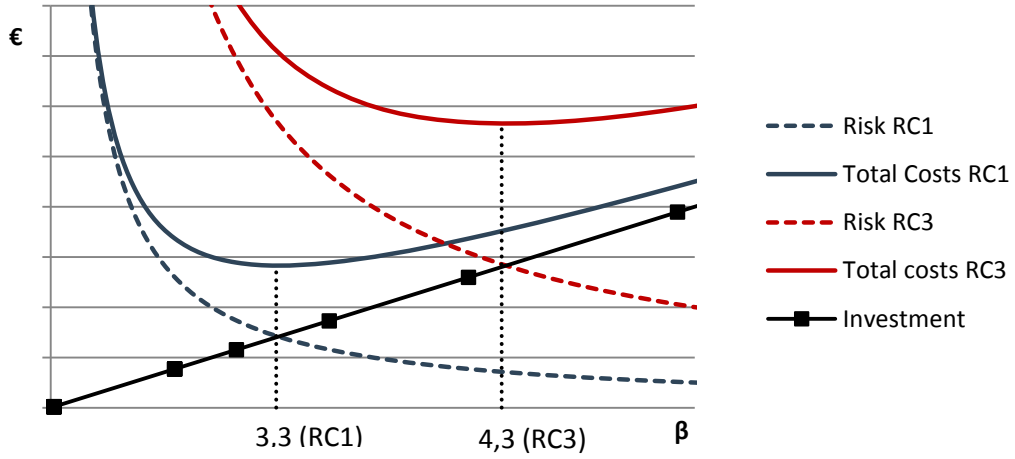


Figure 2-7: Illustration of determining the reliability index based on risk analyses

2.4.2 Fault tree analyses

Failure of a certain system often includes several underlying elements. A fault tree expresses the elements that contribute to the failure probability of the top event. From the total costs which are discussed in the previous section, a maximum failure probability of the top event can be derived. By the use of a fault tree the required failure probability of the underlying elements can be calculated, this is often quite complex due to correlations.

2.4.3 Probabilistic engineering or design

After the required failure probability (or reliability index) is determined for a certain structure, the structure should be designed based on the required reliability. This principle of designing is called probabilistic design.

The reliability of an element (or structure) depends on the margin between the resistance to failure and the loads. The way this margin is calculated can differ by case. In the structural domain, the joint committee on structural safety [18] proposed a level-classification of the calculation methods. In the structural domain it is common to handle a level-classification of calculation methods:

- Level III: These methods calculate the probability of failure by considering the probability density function of all strength and load variables. (Fully probabilistic)
- Level II: This level entails linearizing the reliability in a design point. Furthermore, it approximates the probability distribution of each variable by a standard normal distribution. (Fully probabilistic with approximations)
- Level I: At this level, no failure probabilities are calculated. It is a design method, that considers an element sufficiently reliable if a certain margin is present between the characteristic values of the strength and the loads. This margin is created by taking into account so-called partial safety factors in the design. (Semi-probabilistic)

Level III (Monte Carlo)

The best-known level III method is called the Monte Carlo analysis. This is a fully probabilistic calculation. In this method, the element is written as a reliability function. This function, described in Equation 2.7, implies that if Z is smaller than 0, the element will fail:

$$Z = R - S \tag{2.7}$$

In which:

Z = Reliability function
R = Strength
S = Loads

After defining the variables and their distributions the simulations can start. In these simulations the variables are randomly chosen based on their distributions. In each simulation the Z-value will be calculated. Eventually the failure probability is calculated by dividing the number of simulations for which the Z-value was smaller than 0 by the total number of simulations.

Level II (FORM)

The level II method also known as the FORM (First Order Reliability Method) is well-known probabilistic method. The mathematical theory is explained in Section 7.2.1. The disadvantage of the FORM analysis is that all the distributions are defined as normal distribution. Non-normal distribution should be transformed into normal distributions. For simple reliability functions, this limitation is negligible, but for more complex reliability function it is preferred to use the Monte Carlo analysis. The advantage of the level II method is it directly results in design points. Therefore, if the interest is not the failure probability of the element, but the design points and the influence coefficients of the variables, then the FORM analysis is the proper method to use.

Level I

The essence of this method is the application of partial safety factors. This method is implemented in the standards for structural design. Standards offers partial safety factors for the most common strength and load parameters. These partial factors should be used on the characteristic values of the strength and load. The characteristic values are often the 95% of the upper limit of the load and 95% of the lower limit of the strength, see also Figure 2-8.

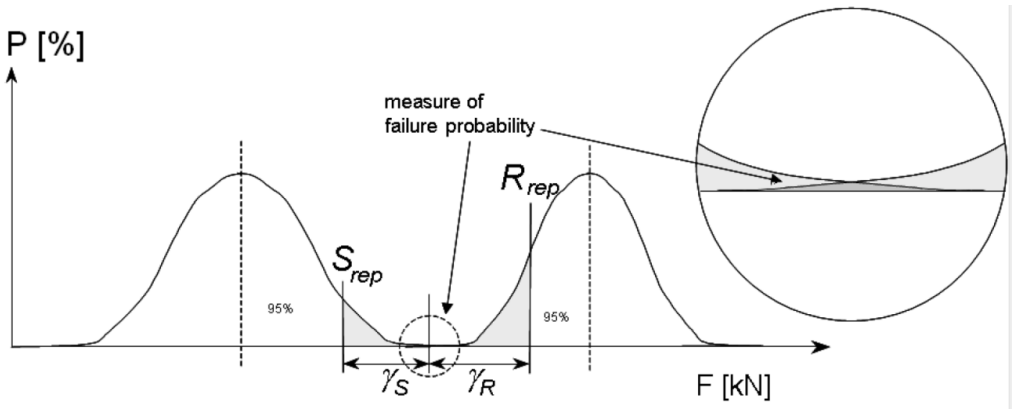


Figure 2-8: Design method Level I

Partial factor are defined as:

$$\gamma_R = \frac{1 + k_R \cdot V_R}{1 + \alpha_R \cdot \beta \cdot V_R} \tag{2.8}$$

$$\gamma_S = \frac{1 - \alpha_S \cdot \beta \cdot V_S}{1 + k_S \cdot V_S} \tag{2.9}$$

in which γ_R and γ_S are the partial factors for the strength and loads respectively. The k_R and k_S indicates the limit of the representative value. For a lower limit of 95% (strength) the value for k_R will be 1.65. The α_R and α_S are the parameter which indicate the influence. The strength coefficient is 0.8 and for the load coefficient is 0.7. The V_R and V_S are the variance coefficients for the strength and loads respectively.

Calibrations to determine the partial factors

The fundamentals of the partial factors that are published in the standard are based on Level II and Level III calculations. These are called calibrations. For several cases the probabilistic calculations are carried out and for each case a series of partial factors, variance coefficients and influence coefficients are derived. If enough results are obtained, a reliable standardized partial factor can be found. This process is carried out for every partial factor presented in the standard. The process of elaborating the partial factors is illustrated in Figure 2-9

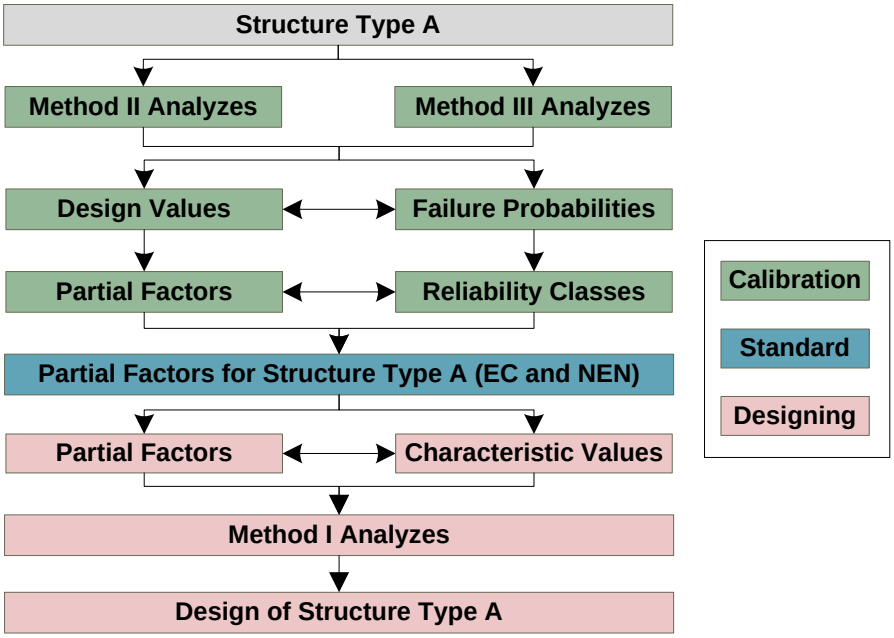


Figure 2-9: Process of elaborating the partial factors in the standards

3 Flood Safety

3.1 Verification and Designing of Flood Safety

Approximately half of the Netherlands lies below sea level and is protected against flooding by primary dikes and other water-retaining structures. Extensive systems of drainage channels and storage basins with associated water-retaining structures and pumping capacity ensure artificial management of surface and groundwater levels. In the middle ages, dikes were designed at the highest known storm surge plus one meter additional freeboard. Today, a probabilistic approach is applied.

A distinction should be made in designing and verifying a flood defense system. In fault tree terms; the designing method is **bottom-up** and the verification method is **top-down**. These two perspectives are called

- "Dike-ring" approach (NL: Dijkringbenadering) (verification)
- "Dike-section" approach (NL: Dijkvakbenadering) (designing)

A short description is:

- The dike-ring approach is based on a critical failure probability of the entire dike ring. With this method, the dikes and structures are related to failure probabilities. By combining the dikes and structures, a failure probability for the dike ring can be derived. This is not an accessible method for designing, because all elements are involved in one design, which can be quite complex. This method is commonly used as a verification method for the dike-ring. With an accurate indication of the failure probability and consequences, decisions can be made about possible measures to increase the safety of the dike (or structures) or not. In Section 3.2 the dike-ring approach will be discussed more explicitly.

- On the other hand in the Netherlands the dike-section approach is applied for designing the dike and hydraulic structures. This approach is a more practical approach, where each dike section is designed so it should retain a particular water level. This water level is called the design water level. The height of the water level depends on the exceedance frequency of the dike (dike ring). The dike-section approach will be explained more explicit in Section 3.5.

3.2 Flood Defense System

Since 1980 awareness has grown that the probability of exceedance of the design water level, the design frequency, is not a good predictor of the probability of flooding. Normally, the dike crest exceeds the design water level by some measure, thus the probability of overtopping is smaller than the design frequency. But some parts of the dike may already be critically loaded before the design water level is reached. Water logging may lead to slide planes though the dike or piping may undermine the body of the dike, with sudden failure as a consequence. Another danger is that sluices and gates are not closed in time before the high water. In short, there are more failure mechanisms that can lead to flooding of a polder than overtopping. Moreover, the length of the dike, the dike ring, has considerable influence. A chain is as strong as the weakest link. So a single weak spot determines the actual safety of the ring.

The probabilistic approach aims to determine the probability of flooding of a certain area and judge its acceptability in view of the consequence. As a start the entire water defense system of the polder is studied. Typically this system contains sea dikes, dunes, river levees, sluices, pumping stations, high hills, ect. (see Figure 3-1).

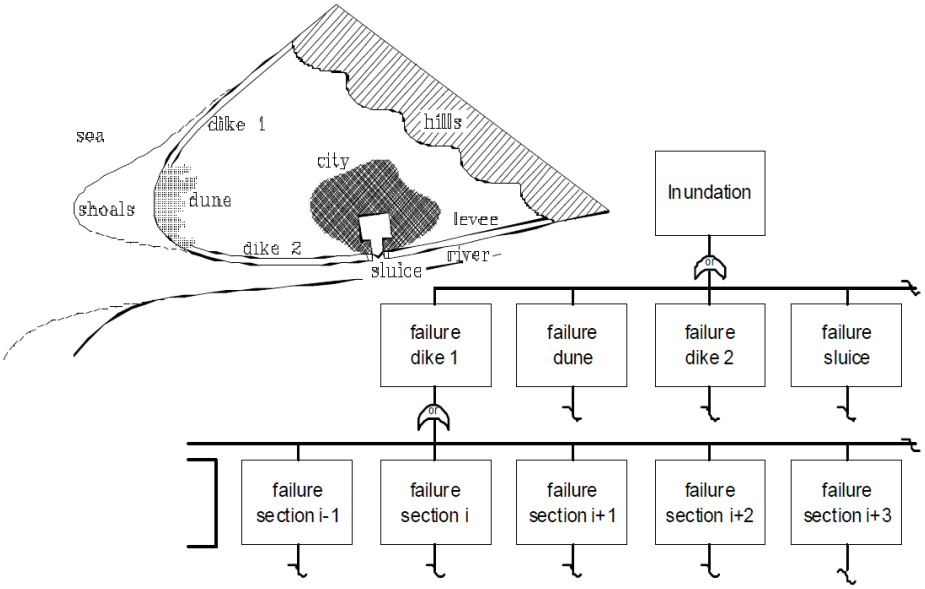


Figure 3-1: Flood defense system and its elements presented in a fault tree [13]

In principle, the failure and breach of any of these elements leads to flooding of the polder. Thus, the probability of flooding results from the probabilities of failure of all these elements. Within a longer element, several independent sections can be identified. Each section may fail due to various failure mechanisms like overtopping, sliding, piping, erosion of the protected outer slope, ship collision, bursting pipeline, ect. The relation between the failure mechanisms in a section and the unwanted consequence can be depicted with a fault tree (see Figure 3-2). The relation between the failure mechanisms for a sluice is shown in Figure 3-6 (section 3.5).

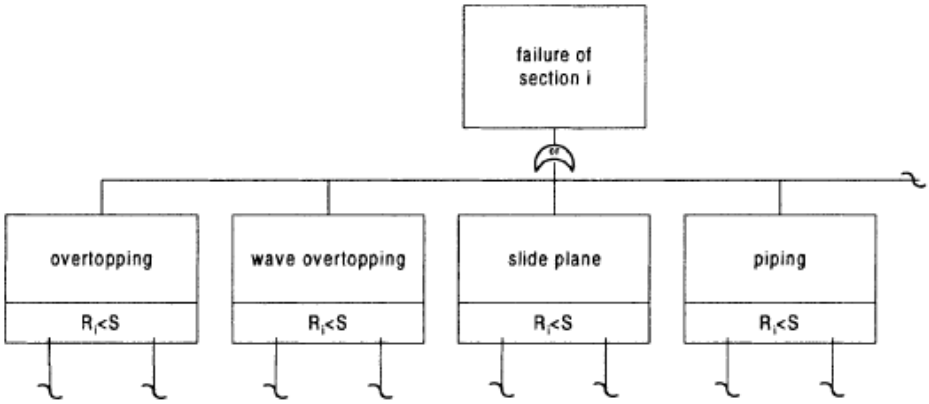


Figure 3-2: Fault tree of a dike section with failure mechanisms [13]

The failure probabilities of the mechanisms can be calculated using the methods of the modern reliability theory such as Level III (Monte Carlo). Correlation between failure modes and correlations between different dike sections have to be taken into account. In the reliability calculations, all uncertainties should be dealt with. All uncertainties are included in the calculations of the failure probability, which is not only a property of the physical reality but also of the human knowledge of the system. By expressing the safety of a dike system as the probability of flooding, the weakest element in the system can easily be found. For example, the results of the calculated probability of flooding of the system are presented in Table 3-1.

.

Table 3-1: Overall probabilities of flooding

Section	Overtopping	Piping	Etc.	Total
Dike section 1	$P_{f,1,overtop}$	$P_{f,1,piping}$	$P_{f,1,etc}$	$P_{f,1,total}$
Dike section 2	$P_{f,2,overtop}$	$P_{f,2,piping}$	$P_{f,2,etc}$	$P_{f,2,total}$
Etc.	$P_{f,etc,overtop}$	$P_{f,etc,piping}$	$P_{f,etc,etc}$	$P_{f,etc,total}$
Dune	$P_{f,dune,overtop}$	$P_{f,dune,piping}$	$P_{f,dune,etc}$	$P_{f,dune,total}$
Sluice	$P_{f,sluice,overtop}$	$P_{f,sluice,piping}$	$P_{f,sluice,etc}$	$P_{f,sluice,total}$
Total	$P_{f,all,overtop}$	$P_{f,total,piping}$	$P_{f,total,etc}$	$P_{f,total,total}$

The last column of the table shows which element or section has the largest contribution to the probability of flooding of the whole system. Inspection of the related row reveals which mechanism will most likely be the cause. Thus, a sequence of measures can be defined that will improve the probability of flooding. It should be mentioned that in Table 3-1, the total values are not equal to the sum of the corresponding failure mechanisms or sections. The correlation between the mechanisms and sections should be taken into account.

3.3 Correlation

In the flood defense system the correlations should be taken into account. There are correlations present between the failure systems and the subsystems. The failure probability of a dike ring can be modelled as a serial failure system. In such a system the correlation between the events play a role in the calculation of the failure probability of the top event. If the probabilities of failure P(E_1) and P(E_2) are of the same order of magnitude, the margin between the lower and upper boundary is also of this order; the margin is thus relatively great. Ditsleven [17] developed an approximation method that can be used to calculate more limited boundaries, between which the probability of failure can be found. This method assumes a known correlation between the failure modes, expressed by a correlation coefficient ρ, and normally distributed reliability functions. He deduced that:

$$P_f = P(E_1 \cup E_2) = P(E_1) + P(E_2) - P(E_1 \cap E_2) \tag{3.1}$$

$$P(E_1 \cap E_2) = P(E_1) \cdot P(E_2|E_1) \tag{3.2}$$

with the upper boundary condition:

$$P(E_1 \cap E_2) = \Phi(-\beta_1)\Phi(-\beta_2^*) + \Phi(-\beta_1^*)\Phi(-\beta_2) \tag{3.3}$$

in which:

E_1 = Failure of Event 1
E_2 = Failure of Event 2
ρ = Correlation between E_1 and E_2
P(A∪B) = Probability of occurrence of A or B
P(A∩B) = Probability of occurrence of A and B
P(A|B) = Probability of occurrence of A, given that B occurs
$\beta_i = -\Phi^{-1}(P(E_i))$
$\beta_1^* = \frac{\beta_1 - \rho \cdot \beta_2}{\sqrt{1-\rho^2}}$
$\beta_2^* = \frac{\beta_2 - \rho \cdot \beta_1}{\sqrt{1-\rho^2}}$

The Ditlevsen method is well in line with the level II failure probability calculations, according to which the correlation coefficient can be calculated by:

$$\rho = \sum_{i=1}^{n} \alpha_{1,i} \cdot \alpha_{2,i} \cdot \rho_i \tag{3.4}$$

in which:

α_{1i} = the influence coefficient for X_i for E_1;
α_{2i} = the influence coefficient for X_i for E_2;
ρ_i = the correlation of the Xi in the event E1 and the Xi in the event E2;
n = the number of basic variables.

In this report, E_1 and E_2 could be understood as two different failure mechanisms, two different locks or a lock and a dike section. These three combinations are used in the analyses of this research. Further on, in Equations 3.4 the correlation between those two items is derived.

3.3.1 Applied correlations in a dike-ring

There are several methods of combining the subsystems and failure mechanisms. This research applies the structure shown in Figure 3-3. In this figure the calculation structure for the failure probability of a dike ring with N locks is expressed. This process starts by calculating the failure probability of each failure mechanism. After that, the mechanisms can be combined to one failure probability, which is an approximation for the failure probability of N locks. Finally the correlation between the locks and the dikes is applied.

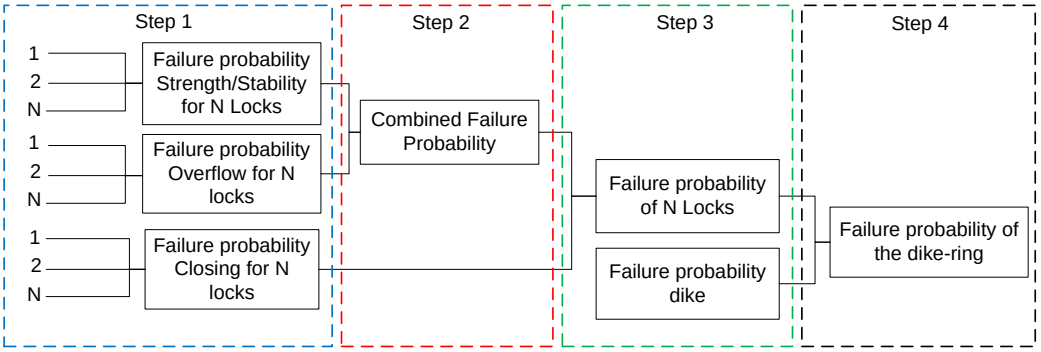


Figure 3-3: Calculation structure for the failure probability of N locks

The correlation between two failure mechanisms depends on the correlated parameter of the failure mechanism. The magnitude of correlation depends of the influence coefficients of the correlated parameter (see Equation 3.4 for the determination). For a failure mechanism the influence coefficients can be found by running a FORM analysis.

The actual correlation parameters are different for each system. It takes a lot of work to find an accurate result for the correlations for a certain system. In this study the correlation parameters will be roughly estimated based on realistic information. An overview of the determination of the correlations has been shown in Appendix I.2. The correlations are based on the influence coefficients for each failure mechanism. Table 3-2 shows the applied correlations between the failure mechanisms for two different locks (i and j). The correlation between the locks and the dike sections has been presented as well.

Table 3-2: Applied correlations in a dike ring (see Appendix I.2)

	Overflow (j)	Closing (j)	Strength (j)	Dike
Overflow (i)	1	0	0,63	-
Closing (i)	0	0	0	-
Strength (i)	0,63	0	0,49	-
Lock	-	-	-	0,55

3.3.2 Length effect

Dikes

If the failure probability for a dike section needs to be defined, the length effect should not be neglected. The length effect increases the probability of failure with the increasing length of the dike. The dike should be seen as a chain, which is as strong as the weakest link. The two main factors determining the magnitude of the length effect are:

- The relative contribution of the load and resistance: A high contribution of the resistance to the reliability of the structure increases the length effect. This is because for flood defenses, load parameters tend to have much larger scales of fluctuation than resistance parameters do.
- The spatial variability in the subsoil: the higher the spatial variability in the subsoil, the higher the length effect (see Figure 3-4)

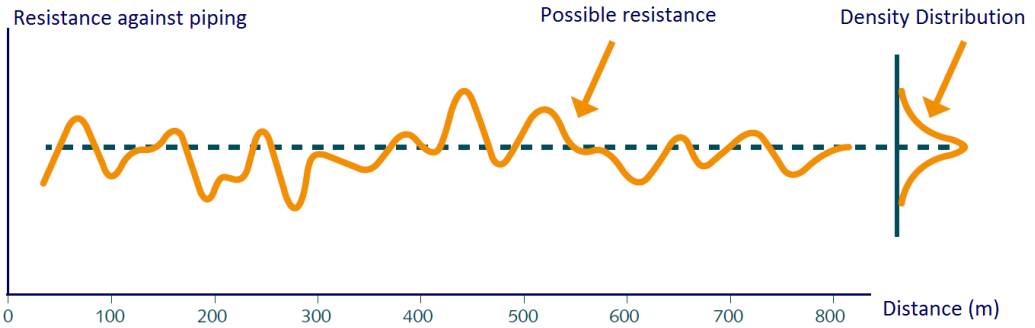


Figure 3-4: Spatial variability of the subsoil concerning the resistance against piping [20]

Hydraulic structures

In contrast with the dikes, hydraulic structures are not continuously dealing with high resistance fluctuations and spatial variability in the subsoil. Thus, the length effect as explained for dikes is basically not applicable. Hydraulic structures could be seen as separate elements in the system that has a certain correlation. In this correlation the influence coefficients for strength and stability play a role, which are α_S and α_R (load (0.7) and resistance (0.8)). Assumed is that the hydraulic structures are fully correlated regarding the loads. By applying the equations 3.4, the correlation can be estimated:

$$\rho(S_i, S_j) = 1, \ \rho(R_i, R_j) = 0, \tag{3.5}$$

$$\rho(Lock_i, Lock_j) = 0.7 \cdot 0.7 \cdot 1 = 0.49 \tag{3.6}$$

In case there are more locks present in a dike ring the reliability of a single lock needs to be higher than the required reliability as is prescribed by the LK. This implies a serial failure system, see Equation 3.1 to 3.3. In Table 3-3 the length effect factors are defined for several numbers of locks. This is a factor which will be multiplied by the reliability index prescribed by the LK (see Table 2-3). In LK a length effect factor of 1.1 is applied [11]. With a correlation of 0.49, this implies that the system is calculated with eight locks.

Table 3-3: The length effect (correlations) for hydraulic structures. The factor is applied on the reliability index for strength and stability. In LK the value of 1,1 is applied

β_{LK} = 4,31 (1/1250)	1	2	4	6	**8**	10
$\beta_{SINGLE\ LOCK}$	4,31	4,46	4,6	4,69	**4,75**	4,79
$\beta_{SINGLE\ LOCK} / \beta_{LK}$	1,00	1,03	1,07	1,09	**1,10**	1,11

3.4 Consequence of Flooding

Estimating the consequences of flooding is a central element in flood risk analysis and management. The totality of flood damage comprises casualties, material and economic damage as well as the loss of or harm to immaterial values like works of art and amenity. However, for practical reasons the notion of risk in a societal context is often reduced to the total number of casualties using as a definition: “the relation between frequency and the number of people suffering from a specified level of harm in a given population from the realization of specified hazards”. If the specified level of harm is limited to loss of life, the societal risk may be modeled by the frequency of the exceedance curve of the number of deaths, also called the FN-curve. The consequence part of a risk can also be limited to the material damage expressed in monetary terms. It should be noted however, that the reduction of the consequences for either measure may not adequately model the public’s perception of the potential loss. The simplification clarifies the reasoning at the cost of accuracy. Nevertheless, for practical tractability, three criteria are defined:

- Individual risk
- Group risk
- Economical risk

The first two belong to the category of “loss of life” criteria, which are often considered as boundary conditions providing a minimum protection level. While individual risk refers to the probability of the death, of an individual person in a specific location, group risk refers to large numbers of fatalities in one event. Economical risk refers to the direct and indirect economic consequences of a disaster, allowing for a direct comparison of investments in and effects of prevention in monetary terms. This model will be applied further on in this report. Table 3-4 shows an example of the economic damage from a flood risk assessment in the Netherlands (VNK [21]).

Table 3-4: Economic damage for some dike-rings in the Netherlands (from VNK [21])

Nr.	F_{exd}	Name of the Dike-ring	Economic Damage (€·10^6)
3	1/2000	Terschelling	160
7	1/4000	Noordoostpolder	9.000
10	1/2000	Mastenbroek	1.200
13	1/10000	Noord-Holland	58.000
14	1/10000	Zuid-Holland	290.000
15	1/2000	Lopiker en Krimpenerwaard	10.000
16	1/2000	Alblassenwaard en Vijfheerenlanden	19.000
25	1/4000	Goeree-Overflakkee	3.700
32	1/4000	Zeeuwsch Vlaanderen	14.000
36	1/1250	Land van Heusden/De Maaskant	18.000
38	1/1250	Bommelerwaard	2.600
41	1/1250	Land van Maas en Waal	6.400
42	1/1250	Ooij en Milingen	1.000
43	1/1250	Betuwe, Tieler en Culemborgerwaarden	18.000
48	1/1250	Rijn en IJssel	6.800
52	1/1250	Oost-Veluwe	3.100

3.5 Flood Safety in the Netherlands

In the Netherlands the dike-section approach is used to design dikes and hydraulic structures. In contrast with the dike-ring approach, the dike-section approach is prescribed by the law (Waterwet/LK). As already mentioned there are 95 dike rings in the Netherlands. Each dike ring is assigned an exceedance frequency. The level of possible damage inside the dike ring determines in which category of frequency the dike ring will be placed (based on flood risk analyses). In the Netherlands applies several frequencies, but a distinction is made in four main categories. Each category describes the minimal exceedance frequency:

1. 1/1250 [1/year]
2. 1/2000 [1/year]
3. 1/4000 [1/year]
4. 1/10000 [1/year]

Dikes

With this approach the correlations between the dike section and the structures, as well as the failure mechanisms, are neglected. The dike sections will be independently designed based on the design water levels (see Figure 3-5). The consequence of this approach is ignorance of the overall safety of the dike ring (the failure probability). In the Netherlands, a project called VNK started to examine the failure probabilities of the dike ring (which is actually a dike-ring approach).

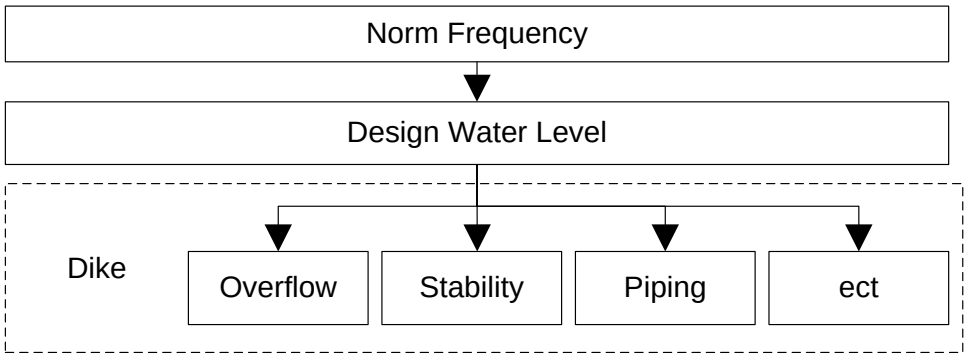


Figure 3-5: In the dike-section approach dikes a designed based on design water levels

Hydraulic structures

LK explains how hydraulic structures should be designed according the dike-section approach. Regarding hydraulic structures, requirements are set on three core failure mechanisms. These are:

- Overflow ➔ Exceedance frequency
- Reliability closing ➔ 0.1 x Exceedance frequency
- Strength and stability ➔ 0.01 x Exceedance frequency

In contrast to dikes, hydraulic structures are linked with a failure probability, excluding the failure mechanism overflow. The design regarding overflow is based on a water level corresponding with a certain exceedance frequency (as for the dikes). But the failure mechanisms “reliability closing” and “strength and stability” are translated into a failure probability. In the upcoming sections, each of the mechanisms will be explained in more details.

In the Basis Report for Water Retaining Structures [4] a fault tree was published for hydraulic structures (see Figure 3-6). The three core mechanisms are expressed with the corresponding failure probabilities. The mechanisms can be elaborated in sub mechanisms. The top event of the fault tree represents failure of the structure. It can be observed that the mechanism overflow plays a dominant role in the overall failure probability. With the fault tree analysis, a Monte Carlo analysis can be performed. Such analysis gives the proportions of the failures. For overflow an influence value of 90.1% is found (of 1000 failures, 901 are caused by overflow). The other mechanism, "strength and stability", plays a minor role in the fault tree (0.9%).

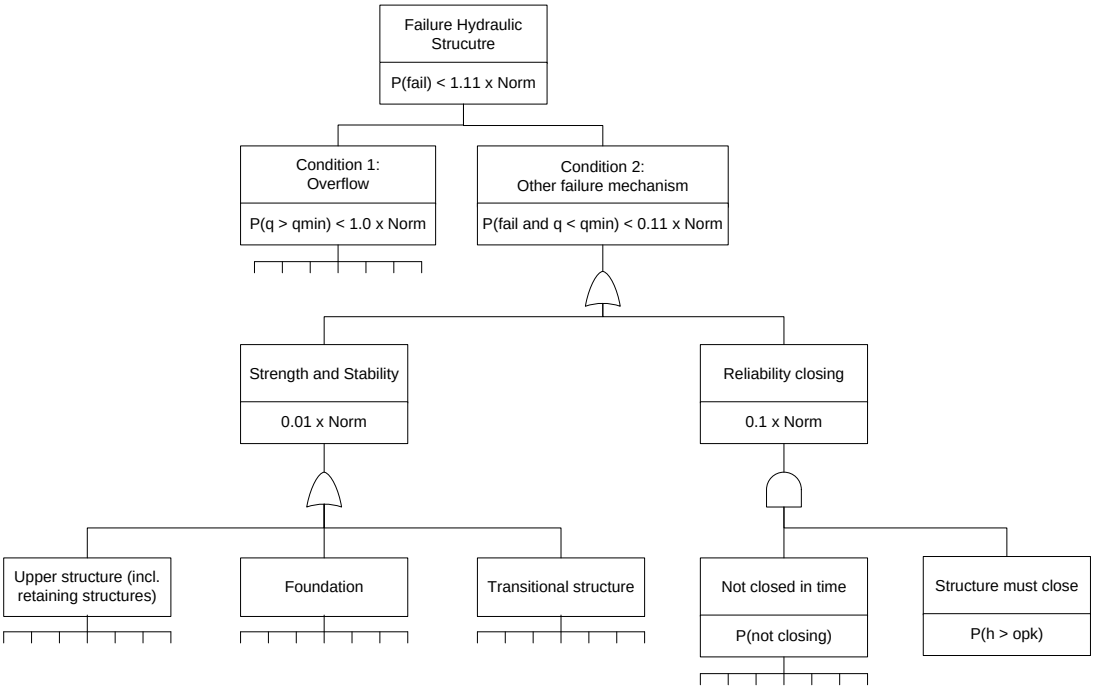


Figure 3-6: Fault tree of hydraulic structure according dike-section approach (4)

3.5.1 Overflow

The overflow refers to the permissible flow over the lock gate. With this flow rate, the retaining height of the lock can be calculated. The retaining height refers to the top of the lock gates. There are two critical requirements concerning overflow. These are:

- Sufficient capacity behind the structure to store the overflowing water;
- The high velocities created by the overflow, should not bring the stability of the structure in danger.

The last requirement is in most situations not an issue. The acceptable overflowing water depends on the area behind the structure. In case of high water many fields are flooded in order to extend the storage capacity. The acceptable overflow can be transformed into an acceptable overflow height, called h_{ovl}. The total height of the gate can be defined as follows;

$$h_{top} = MHW + h_{hwr} + h_{wind} + h_{set} + h_{saf} - h_{ovl} \quad (3.7)$$

in which:

h_{top} = Height of the gate of the structure [m NAP]
MHW = Mean high water level [m NAP]
h_{hwr} = Additional high water height [m]
h_{wind} = Wind set up [m]
h_{set} = Settings of the structure [m]
h_{saf} = Safety height [m]
h_{ovl} = Overflow/Overtopping height [m]

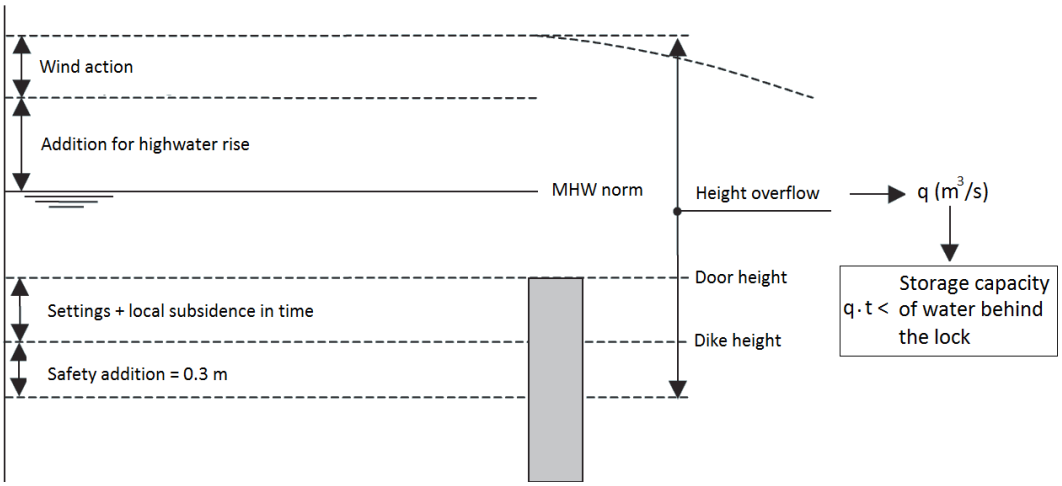


Figure 3-7: Overview determination of top level of the gate [3]

The principal of this type of designing correspond with the designing method for dikes. The height of the gate does not correspond with a reliability or failure probability, but with a design water level (MHW). These design water levels are derived from the exceedance frequencies.

3.5.2 Reliability closing

If the water reaches a certain water level, the gates of the structure needs to close. This water level is called the closing point. The closing point depends on the storage capacity behind the structure. If the inflow (V_{open}) reaches the critical inflow ($V_{acceptable}$) the gate should close. The critical inflow can be transformed into a critical height, the closing point. The failure probability should be approached as:

$$P_f = P\{V_{open} > V_{accepetable}\} = P_{nc} \cdot N\{V_{open} > V_{accepetable} | nc\} = P_{nc} \cdot n_j < 0.1 \cdot F_{exd} \quad (3.8)$$

In which:

$P_f(\ldots)$ = Failure probability of the discussed element [1/year]
P_{nc} = Probability of failure of the closing process (1/closing required)
n_j = Frequency of exceeding the closing point (closing required/year)
F_{exd} = Exceedance frequency of the related dike section [1/year]

For the mechanism closing a fault tree is represented in Figure 3-8. The fault tree includes an AND-port. This implies that both conditions should happen to ensure the failure of the system. One of these conditions it that the closing point should be reached. From that point, the gate should close. The second condition implies failure of closing the gates. This condition is divided into a couple of possible causes. In LK, an explicit explanation is given for an accurate approximation of the failure probability due to closing.

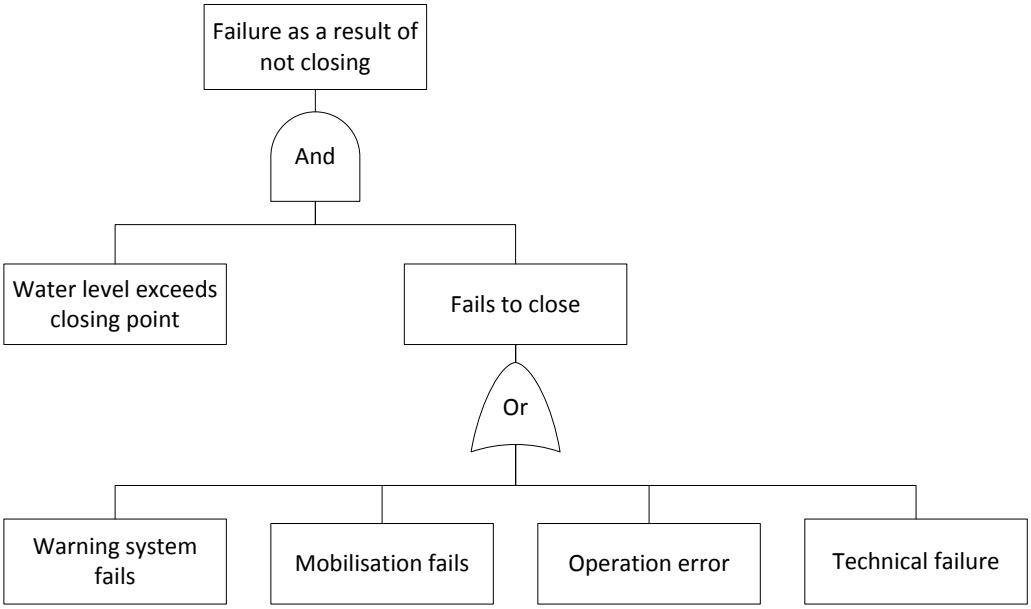


Figure 3-8: Failure tree for reliability of closing [3]

3.5.3 Strength and stability

For the strength and stability of a structure, the LK prescribes a reliability which is more severe than the other failure mechanisms. According to the LK the probability of failure for the strength and stability is expressed in equation 3.9. The equation gives a certain failure probability for the condition that the overflow is lower than the critical overflow. This means that the construction should not fail on strength and stability and should also not fail on overflow. If the structure fails due to overflow, it does not suffice to the requirement.

$$P\{failure \mid q < q_{accept}\} < 0.01 \times F_{exd} \quad (3.9)$$

in which:

$P\{A|B\}$ = Failure probability of event A, given the fact that event B occurs
q = Inflowing water [m^2/s]
q_{accept} = Acceptable inflowing water (according overflow) [m^2/s]
F_{exd} = Exceedance frequency of the related dike section [1/year]

Failure of the structure as a result of lack of strength and stability will not always cause flooding. There are a couple of failure mechanisms that will lead to flooding. These failure mechanisms are:

- Piping
- Overall stability lock head
- Failure of the gate

Piping
Piping is the process of (ground) water flow, in the soil under and/or around the structure, having enough transport capacity to erode soil particles. As soon as the first particles are carried away, more water will be able to flow along the same route because of the reduced flow resistance and more soil particles will be washed away. The process strengthens itself, and a pipe develops. This will undermine the structure and, in the end, result in instability of the whole structure.

Overall stability
During extreme water, the lock stability could fail. Failure of stability will result in a certain movement of the lock gate and eventually fail to retain the water, with flooding as a consequence. There is a distinction between the three types of stability: horizontal stability, vertical stability and overturning.

Gates
The gates are the elements of the lock that stands between the two water levels. If the gate fails completely the water will flow into the protected area. Failure of the gate depends in most situations of more than one element. For simplicity, the required reliability of the gate applies to all the elements of the gate.

For these failure mechanisms, partial factors are used to ensure the required reliability. For structural calculations the NEN (now the EC) standards provides the partial factors. However, in the NEN (EC) there is no proper approach for hydraulic loads. For hydraulic loads, the LK provides several partial factors. Figure 4.6 lists the relevant loads for a hydraulic structure. This table is published in the LK. It shows the partial factors for each load and the corresponding standard. In the table the partial factors from the TGB (NEN) are given. The factors for the hydraulic loads, in particular the partial factor for water level differences, are more interesting. In Appendix A an explicit list of the relevant partial factors for LK and NEN/EC is given.

4 Design of Locks

4.1 Position of Designing in the whole Life Cycle

Designing a structure is more than just the technical calculations. It is a small part of much large cycle, the life cycle. In this cycle, the development of a structure is illustrated over time from the stage the idea was generated through the demolition stage. In Figure 4-1 the stages in the life cycle are presented.

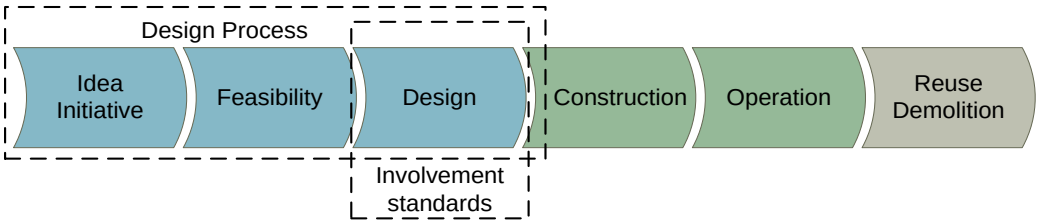


Figure 4-1: Life cycle [27]

Within the design process, three main stages can generally be distinguished. In the first initiative stage there is an actual and/or future problem that triggers a stakeholder, the government (national or local), pressure group or sometimes an individual visionary, to define objectives for improvement of the present and future situation at a macro system level.

In the following feasibility stage research, is undertaken to identify all the stakeholders that may be involved, their objectives, the functional requirements to possible solutions and whether or not there are solutions for which costs (incl. risks) and benefits (incl. other values) are in balance, not only for the near future but for the whole expected lifetime. The feasibility stage ends with a go/no-go decision.

In the technical design stage, these first solutions or even better, sketch or conceptual designs, will be elaborated to lesser or greater extent (from preliminary design via final design to detailed design). In this stage the standards starts influence the design. The dimensions will be defined and the amount of reinforcement can be named. For this research, this stage will be interesting.

4.2 Scope of Hydraulic Structures

In this report, the gap between the Eurocode and hydraulic structures will be researched. This means the results will be interesting for hydraulic structures in general. However, hydraulic structures have quite a large scope. To make this research more approachable, the decision was taken to focus on one type of structure. For this research the structures are limited to locks. The main reason for this is the important role locks have in the flood defense system in the Netherlands. Locks can be divided in a variety of types. A list of those types follows;

- Dewatering gate
- Stop lock
- Guard lock
- Navigation lock

The most common type of lock is the navigation lock, which is why this type is considered within this research. A navigation lock, referred to as lock from here on, is the link between two sections of a waterway (river, sea or channel) with different water levels. The lock enables the transfer of a ship from one section to another in the waterway. Obviously, the water level difference between the two sides of the lock, the lift, is one of the main parameters to be considered in the design of the lock. There is a large variety in the types of gates. The three types that can be found frequently are: mitre, rolling and lift gate. Which type will be preferred, depends from the boundary condition.

4.3 Navigation Locks

A navigation lock is part of a lock complex. This is the overall name for all the structures that contribute to the functioning of the lock. Within the lock complex, three parts can be distinguished in a functional sense: lock approach, leading jetty and chamber. The lock approach is the navigation area between the connecting waterway and the lock complex, where approaching vessels have the opportunity to decrease speed and moor to a guiding structure if necessary. With this, sufficient view and overview should be ensured both day and night. The lock approach should therefore be free of obstacles and not be situated in a bend. The leading jetties form the transition area between the lock approach and the chamber and are meant to guide the vessels, by both mechanical and visual means, quickly and without damage, into the chamber from either the mooring area or from moving navigation. In the chamber the vessels are tied up and the water leveling process starts.

4.3.1 Elements

The main objects in the design of a lock complex are shown in Figure 4-3 in the form of an object tree. In this research, the focus is limited to the lock and the lock approach. The control tower and lock site will not be discussed. These are not by definition hydraulic structures. The lock and the lock approach can be divided in several sub objects. The lock can be divided in the chamber and the heads. Again, these elements can be divided in even smaller elements. The same principle follows for the lock approach. Figure 4-2 is limited to two levels of elements.

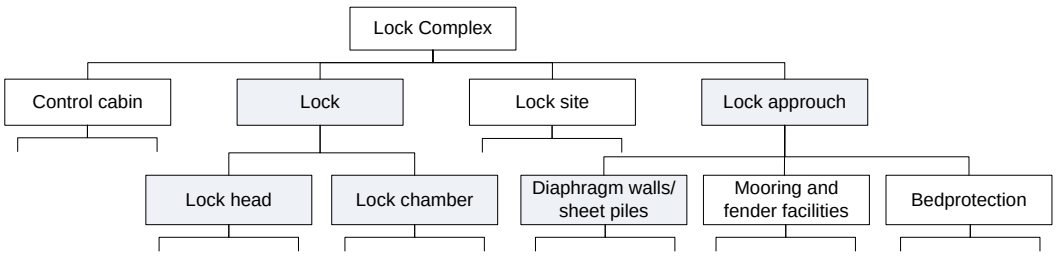


Figure 4-2: Tree of objects concerning a navigation lock

4.3.2 General costs of a lock

Later in this report, a case study will be carried out. For that case study, a project of Grontmij is used. Insights into the project costs were available. The project is about a design of a second (new) lock in the lock complex in Eefde. This second lock will enhance the capacity of the lock complex. The design, made by Grontmij, contains a couple of elements that are relevant for a rough insight into the costs. These elements are:

- Lock gate
- Lock head
- Lock chamber
- Lock approach
- Mechanics and electrics

The costs have been calculated with the program GWW Calc. This calculation can be quite accurate if sufficient information is available. In Figure 4-3 an overview is shown of the investment and lifetime costs.



Figure 4-3: Overall costs of a navigation lock (Example: Eefde)

Initial costs

From the table it is clearly visible that the initial costs of the lock head and the lock approach (IJssel) are critical for the total estimation if one is looking at the Civil Engineering items. Those elements can be divided in smaller elements. For the lock head, there are three elements that play a major role in the costs. Those are; the construction pit (25%), concrete U-box (24%) and the upper structures (37%). In Section 5.5 the first two elements and the corresponding costs will be clarified. The last element, the upper structures is excluded from the case study. The elements that are part of the upper structures are with regard to the structure not specifically related with the navigation lock. The lock approach is the area before and after the lock. It contains, for example, guiding walls and waiting areas. The high costs arise from the large amount of steel that is required for the combi-walls.

The initial costs also include the costs required to construct the structure. With GWW-Calc, the construction costs are calculated by taking a certain percentage of the investment costs. Risks and profits are all included.

Lifetime costs

Figure 4.5 indicates that the mechanics/electrics and the head has a high influence on the cost during lifetime. For the gate the lifetime costs are higher than the investment costs. This can be clarified by the high cost for replacing structural parts. The mechanical system that, opens and closes the gates contains elements that are more vulnerable and should be replaced more frequently. The same applies for the gates. During the lifetime of the navigation lock, the gates should be replaced two times.

If the current value of the expenses for the present need to be calculated, there are two factors that plays a role; the inflation and the interest determine the current value of future expenses. The formula is expressed in the equation below:

$$I_0 = I_t \cdot (1+i)^t/(1+r)^t \tag{4.1}$$

in which:

- I_0 = Present value of investment (at t = 0) [€]
- I_t = Required investment value in the time of investment (t = t) [€]
- i = Inflation rate [-]
- r = Interest rate [-]
- t = Time [years]

4.4 Functions of a Lock

A lock is built for navigation. Therefore, the functional requirements related to navigation are of primary importance: transferring vessels from one water level to another. The other functional requirements (related to flood control structure, water management and dry infrastructure) come into view when considerations start on fitting the lock into the environment. A summation of the functions of a lock is:

- the navigation function
- the water retaining function
- the water management function
- the cross waterway connection

The two main functions are more explained more explicitly in the following sections.

4.4.1 Navigation function

In the Netherlands there is a classification made for vessels (CEMT-class). Every lock is designed on the basis of a maximum allowed vessel. This classification will determine the global dimensions of the lock. Regarding the navigation function, it is important to achieve the largest availability for locking of vessels. This cannot always be controlled. In some cases, the unavailability will be for a short period (heavy cross wind). However, in principle, prolonged malfunctions (up to several days) usually caused by collisions and maintenance, are more serious than shorter disruptions to which navigation is generally able to adjust with more flexibility. In planned prolonged obstruction (maintenance), it should be considered whether this could be executed in periods of less traffic (weekends, season). Not all obstructions lead to complete unavailability. Sometimes, navigation is able to pass within certain limitations.

4.4.2 Water retaining function

The Netherlands is country that needs protection against flooding. Navigation locks are part of the system that protected the inland against flooding. For locks, a distinction can be made in primary and secondary navigation locks. The difference between those is that a primary lock is part of the primary flood defense. It is part of a system that protects a large area against flooding. As already mentioned, such a system is called a dike ring. It is a ring around a large area that will be protected. If a lock in the primary system fails, the area behind it will flood (if no measurements are taken). A secondary lock lies inside a dike ring. It protects a much smaller area. In the Netherlands, there are in total 95 dike rings (see Figure 4.4 for dike rings in the Netherlands).

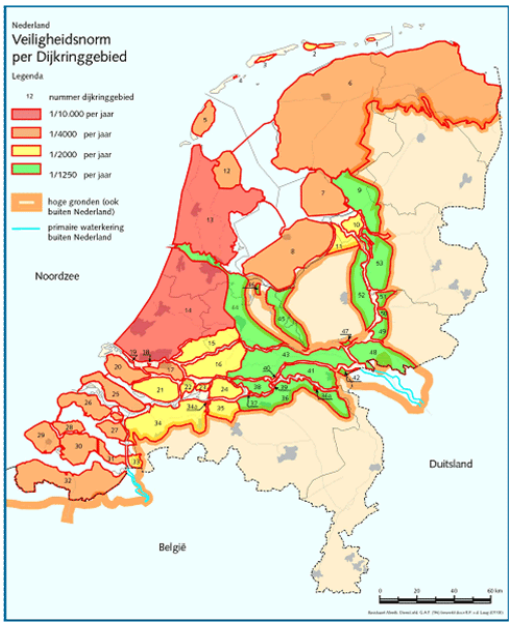


Figure 4-4: Dike rings in the Netherlands

4.4.3 Relation function with failure of elements

For a lock complex, not every element should be designed on basis of the same standards. Failure of one of the elements in a lock complex will not always have the same consequence. In other words, in the situation one of the elements fails, it does not automatically mean that all the functions of the lock are disabled. In Figure 4.5 a relation is illustrated between the elements and the two main functions of the lock. The relation stands for: *failure of element x disables the function to which it is linked in the figure*. From the figure it can be seen that the function water retaining is just related to three elements. The function locking is related to every element in the complex. That would be logical, because a failed element should be repaired. Repairing of any element will lead to complete or partial unavailability of the lock. However, for failure of the lock chamber for example, the lock will still be able to retain the water.

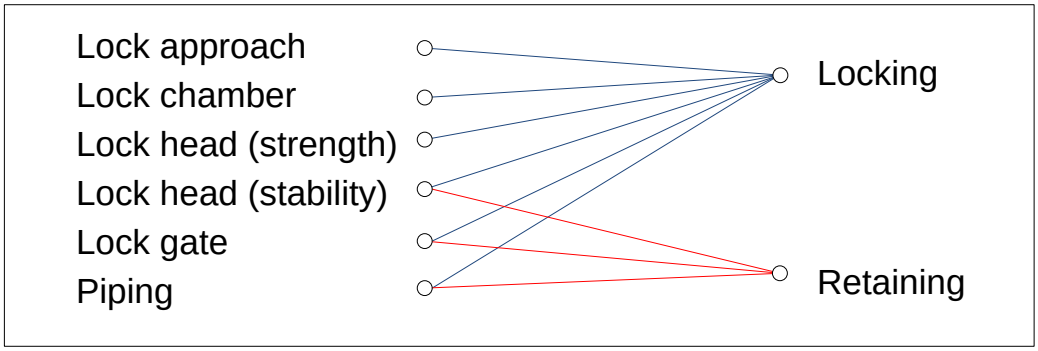


Figure 4-5: Relation failing element with influence on function

Therefore, each element should be designed with the right standards. In the Eurocode, a distinction is made in reliability classes (see 2.2.1). There is a class that stand for high consequences (RC3) and a class with medium consequences (RC2). Failure of the function water retaining, will lead to flood, which has high consequences. The elements which cause this, should meet the requirement of reliability class 3. Failure of the function locking (navigation) has much lower consequences. Thus, these elements should be designed according reliability class 2. Basically, a connection can be made between the different elements in a lock complex and the required reliability. This distinction is presented in Table 4-1. It would be recommended to design a lock based on this subdivisions. This is not an ordinary thing to do. It requires some good arguments to make a distinction as is presented in Table 4-1. If this kind of decisions is made, an Engineering Firm should ask for approval from their clients.

Table 4-1: Distinction in reliability between the elements

Function	Element	Reliability class
Locking	Lock approach Lock chamber Lock head (strength)	RC 2
Retaining	Lock head (stability) Lock gate Piping	RC 3

4.5 Standards and the corresponding partial factors

In the preceding sections it was mentioned that the NEN/Eurocode prescribe a semi-probabilistic approach for structural designing (Strength and Stability). This method utilizes partial factors for the loads and strengths of the structure. The same applies for the Leidraad Kunstwerken. Apart from requirements concerning closing and overflow, the LK prescribed also requirements regarding strength and stability. These requirements are only limited to the load partial factor for water level differences. For the other partial factors, the LK refers to the NEN/Eurocode. This distinction in requirements is illustrated in Figure 4-6.

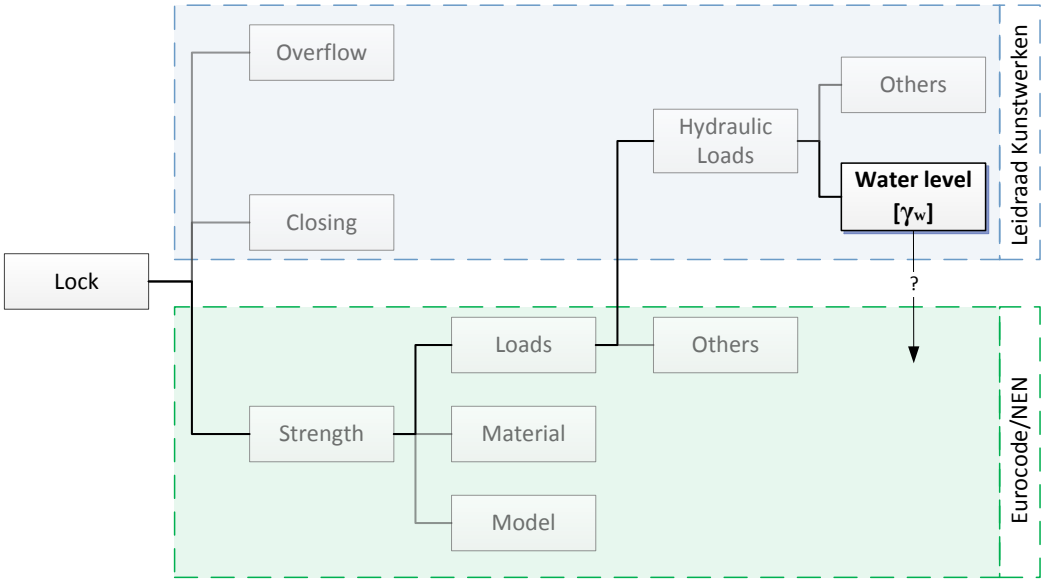


Figure 4-6: Distinction in partial factors in the period of NEN (till April 2012)

Apparently, the LK does only prescribe partial factors for hydraulic loads (and in particular the partial factor for water level differences). This is directly where the main problem of this research emerges; the partial factor for water level differences. This is the level where Eurocode and LK crossing paths. There are more hydraulic loads prescribed in the LK, except these loads are not really influential. There is no calculation method published for the other hydraulic loads in LK. In all the other components (in the framework of strength and stability) no conflicts emerge.

The conflict regarding the partial factor for water level differences arises from the applied reliability index. In Chapter 2 is explained that the LK and Eurocode prescribes a different reliability index concerning the strength and stability for locks that are part of the flood defense system. This resulted in the discussion of which reliability should be used. The law prescribes the Eurocode, but the LK is based on a more corresponding theoretical framework. This discussion is illustrated in Figure 4-7.

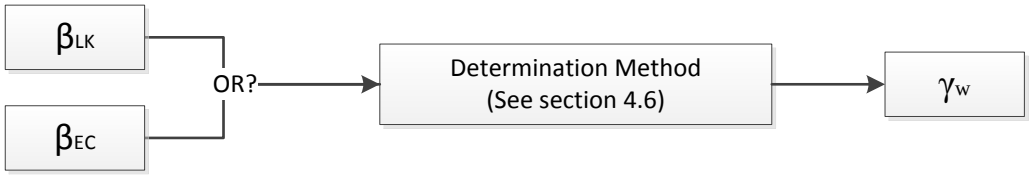


Figure 4-7: The discussion which was created since the Eurocode became valid. The discussion say: Which reliability should one apply, LK of Eurocode?

4.6 Determination Method of the Partial Factor for Water Level Differences

In the previous section (4.5) it is mentioned that the involvement of the LK (concerning strength and stability for hydraulic structures) lies in the partial factor for water level differences. It is also mentioned that the determination method of this partial factor involves the reliability index. In this section the determination method will be explained more specifically and also the influence of the applied reliability index on the magnitude of the partial factor is clarified.

In Chapter 2, Table 2-3, the classifications of the Eurocode and the LK were listed. The Eurocode considers three classes, which resolves in three reliability indexes. For LK at the other hand, this is approached differently. The required reliability for the structure depends on the exceedance frequency of the corresponding dike-ring; see the expression below:

$$\beta_N = -\phi^{-1}(\xi_c \cdot f_N \cdot F_{exd}) \tag{4.2}$$

in which
- β_N = Reliability index of the structure in N years [-]
- $\phi(\ldots)$ = Inverse of the standard normal distribution [-]
- ξ = Failure space according LK (which is 0.01) [-]
- f_N = Reference period [years]
- F_{exd} = Exceedance frequency of the corresponding dike [1/year]

In Table 4-2 the reliability index in relation with the frequency of exceedance are expressed for different reference periods. In these tables there is a column added with the reliability index in case the length effect plays a role. By increasing the reliability index by 10 % (= 1.1 · β) the length effect is included in the reliability of the structure. In a dike ring several locks are present. The number (1.1) is generated from the correlation between those locks. The more locks are present, the higher the number should be (see for an explanation 3.3.2 [11]).

Table 4-2: Reliability indexes in relation with exceedance frequency

Frequency	β_1	l.e.*	β_{10}	l.e.*	β_{50}	l.e.*	β_{100}	l.e.*
1/1250	4,31	4,75	3,78	4,15	3,35	3,69	3,16	3,47
1/2000	4,42	4,86	3,89	4,28	3,48	3,83	3,29	3,62
1/4000	4,56	5,02	4,06	4,46	3,66	4,03	3,48	3,83
1/10000	4,75	5,23	4,26	4,69	3,89	4,28	3,72	4,09

*l.e. stands for length effect

Now the reliability index is defined, the next step is to calculate the load partial factor. The determination method is explained in LK. There are two possible methods to compute the load partial factor:

- Detailed method
- Simplified method

The *simplified method* is basically similar to the detailed method, only some parameters in the equation are standardized. This will be clarified in section 4.6.2. First the complete determination method (the so called *detailed method*) will be explained.

4.6.1 Detailed Method

The detailed method is in line with the design Method I from section 2.4.3 (Semi-Probabilistic). The determination method is based on a design water level and a characteristic water level. For the characteristic water level usually the water level which corresponds with the exceedance frequency is used (which is called the Mean High Water level (MHW)). The design water level depends on the reliability which is required (β). The detailed method is expressed in equation 4.3 and 4.4.

$$\gamma_w = \frac{u - B \cdot \log(\Phi(\alpha_s \cdot \beta_N) / f_N) - h_b}{u - B \cdot \log(F_{exd}) - h_b} \cdot e^{\{\alpha_R(\beta_N - 3{,}6)\cdot\sqrt{\ln(1+V_R{}^2)}\}} \tag{4.3}$$

which can be rewritten as:

Design Value

$$\gamma_w = \frac{1 - \frac{B}{u'} \cdot \log(\Phi(\alpha_s \cdot \beta_N) / f_N)}{1 - \frac{B}{u'} \cdot \log(F_{exd})} \cdot e^{\{\alpha_R(\beta_N - 3{,}6)\cdot\sqrt{\ln(1+V_R{}^2)}\}} \tag{4.4}$$

Material Correction

Characteristic Values

in which:

B	=	Decimeringshoogte (see Appendix B) [m]
u	=	Mean water level at the outer side of the lock [m]
u’	=	Difference in water level between the inner and outer waters ($u - h_b$) [m]
h_b	=	Water level at the inner side of the lock [m]
γ_w	=	Partial factor for water level differences [-]
α_s	=	Influence coefficient for loads (= 0,7)
β_N	=	Reliability index regarding an reference period N [-]
f_N	=	Reference period [yr]
F_{exd}	=	Exceedance frequency [1/yr]
ϕ(…)	=	Standard normal distribution
α_R	=	Influence coefficient for strength (= 0,8)
V_R	=	Variance coefficient for strength (depends of the material) [-]

In the process of calculating the partial factor, three combinations should be verified. Eventually the critical combination results in the partial factor which should be applied. The three combinations are based on a specific reliability index in combination with corresponding reference period. They are defined as:

1. The reliability index β_{10} (of LK) is used in combination with the corresponding lifetime of 10 years
2. The reliability index β_N (of LK) is used in combination with the corresponding lifetime of N years
3. The reliability index $\beta_{EC/NEN}$ is used in combination with the corresponding lifetime of 50 years

The first two combinations are in line with the LK. The third combination is defined by the reliability index according the Eurocode or previously the NEN. This reliability index depends on the reliability class used for the structure.

In the expression a new parameter is introduced, the *Decimeringshoogte*. This parameter refers to the scale of the concerning Gumbel distributed water level. In Appendix B this parameter will be explained more explicitly. Another important value is the influence coefficient α_S. This value indicates in which extent the concerning load influences the failure probability of the structure. A higher influence coefficient results in a higher partial factor. The current applied influence coefficient (0,7 for loads) are prescribed by ISO.

In addition, a material correction is added to the partial factor. This correction factor is used to compensate the lower reliability which is applied for the materials. This can be explained as follow; the design values for the material properties represent a constant reliability. If the required reliability of the structure is higher than the reliability of the material, a compensation should be applied on the load partial factor (which is the material correction factor form equation 4.3 and 4.4).

4.6.2 Simplified Method

In the *simplified method* the ratio between the decimeringshoogte (B) and the difference in water level (u') will be limited to **1.0**. The theory about the decimeringshoogte will be discussed in Appendix B. This is a limit that can be assumed for the structures in primary flood defense in the Netherlands [11]. With this information an upper limit of the partial factor can be calculated as follows:

$$\gamma_w = \frac{1-\log(\Phi(\alpha_s \cdot \beta_N)/f_N)}{1-\log(F_{exd})} \cdot e^{\{\alpha_R(\beta_N - 3{,}6)\cdot\sqrt{\ln(1+V_R^2)}\}} \qquad (4.5)$$

In Table 4-3 the standardized partial factors are presented. In the table the effect of the reliability class is expressed. A higher reliability class will result in a higher partial factor. In addition, the difference in exceedance frequency is shown. Since the representative value for a water level will be higher for a high frequency, the partial factor will be lower for a higher frequency. Further, the influence of the length effect is clearly shown.

Table 4-3: Standardized partial factors (from simplified method)

	No length effect			With length effect		
	LK	EC-RC2	EC-RC3	LK	EC-RC2	EC-RC3
1/1250	1,20	1,29	1,52	1,26[1,2]	1,29[2]	1,52[2]
1/2000	1,15	1,23	1,45	1,26[1]	1,26	1,45
1/4000	1,09	1,15	1,36	1,26[1]	1,26	1,36
1/10000	1,09	1,09	1,25	1,26[1]	1,26	1,26

1: This factor is rounded to 1,25 and also found in figure 3.10 and 3.11 (published in LK)
2: These unrounded factors are defined in the figures 3.12, 3.13 and 3.14

The numbers which are indicated in red follow from the critical combination 1 or 2. Which means, that for those values the LK is more conservative (more critical) as the NEN or Eurocode. The numbers in black indicate that combination 3 (according NEN or

EC) is critical. This clearly shows that in most situations the Eurocode is more conservative than the LK. This is in contrast with the NEN. In period the NEN was to be used, the LK was for most situations more critical.

Partial factors without the material correction factor

With the Eurocode the material correction factor should probably be adapted. If the correction factor is negligible in the formula, the adaption of it will also lead to minimal changes. At the other hand, if the material correction factor is heavily weighted in the formula, a small change can lead to significant changes in the partial factor for water level differences. In Table 4-4 an overview is shown for the partial factors in case the material correction factor is omitted. In comparison with the previous Table 4-3, it can be concludes that the correction factor plays a significant role in the determination of the partial factor for water level differences.

Table 4-4: Standardized partial factors without material correction factor

	No length effect			With length effect		
	LK	EC-RC2	EC-RC3	LK	EC-RC2	EC-RC3
1/1250	1,20	1,25	1,36	1,25	1,25	1,36
1/2000	1,16	1,19	1,30	1,22	1,22	1,30
1/4000	1,11	1,11	1,21	1,18	1,18	1,21
1/10000	1,07	1,07	1,12	1,14	1,14	1,14

5 Specification of the Research

5.1 Research Questions

This report started with a general problem. According the law every structure that is build should meet the requirements based on the most critical standard. Previously for hydraulic structures the critical standard was Leidraad Kunstwerken (based on the Waterwet). This standard was more conservative than the NEN (based on the Woningwet). Now the NEN is replaced with the Eurocode, a higher safety class is added that is more conservative as the LK. So, hydraulic structures are forced by the law to be designed based on the Eurocode. In the previous section the required theory concerning this problem is discussed. From this theory some more approachable issues are observed. In this research three "issues are tried to solve. In these issues a distinction is made in the influence of the Eurocode and the applicability of the Eurocode. Based on this distinction, three research questions where defined:

Influence of the Eurocode

1. What are the additional costs for a lock design based on the Eurocode, and in which extent do the structural elements influence that? (Chapter 6)

Applicability of the Eurocode

2. In what extent is the current determination method (according LK) for the partial factor for water level differences and those applied parameter, applicable and how should the method be adopted to make it more accurate?
    - **c.** Is the influence coefficient which is prescribed by the ISO and Eurocode used correctly in the determination method (Chapter 7)
    - **d.** Are the reliability classes represented in the Eurocode applicable in the determination method for water level differences? This question asks for more research than this report can offer. For this reason the question is limited and rewritten as: are the failure systems between hydraulic structures and buildings comparable? Therefore, is the investment (from LK to EC) lower as the decrease in risk? (Chapter 8)

This research will continue with three case studies in which the research questions are tried to be answered. A short explanation of the specific contents of these case studies is given in the next sub sections. Finally in Chapter 6, 7 and 8 case studies are carried out.

5.2 Field of research of het Case Studies

In the three case studies, research is done to the influence and applicability of the Eurocode on locks. They are focused on three different aspects of interest. The case studies are all related to locks. The field of which a lock is part of is quite large. In order to get an overview of the relations between the cases studies, Figure 5-1 is presented.

In this figure a simplified research area of a lock is sketched. On a large scale, a lock is related to the corresponding dike ring and on a detailed level it is related to the designing requirements for a lock. The three case studies cover a different part of these areas. Case one is focused on the costs of a lock design, case two deals with the partial factor for hydraulic loads and case number three discussed the flood defense system of which locks are part of.

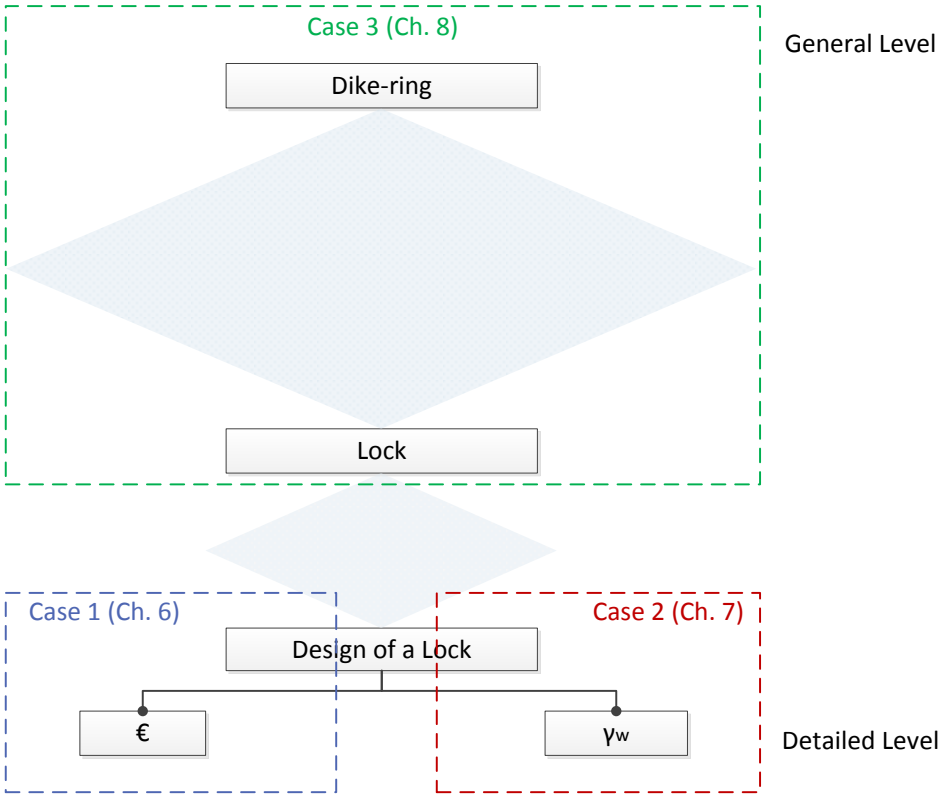


Figure 5-1: Relations between the case studies by means of research areas

5.3 Influence of the Eurocode (Case 1)

In the first case the actual effect of the Eurocode on a design of a lock will be researched. With stricter standards it will be expected that the total costs will rise. But is that really true, and if so, with what kind of differences will this happen? The goal of this chapter is to find a realistic indication of the additional investment, in which the investment is defined as:

$$\Delta I = I_{EC} - I_{LK} \tag{5.1}$$

In which:

ΔI	= Investment costs to upgrade the reliability class
I_{EC}	= Total Project Costs of a design based on Eurocode
I_{LK}	= Total Project Costs of a design based on LK

Besides the range of the additional cost, it is also interesting to know which elements are responsible for these additional costs and if it is possible to reduce them. It is also a useful given regarding the flood risk analyses elaborated in Chapter 8. The investment results in a decrease of the failure probability which subsequently leads to a decrease of the risk. This is explained in Section 5.5 and Chapter 8.

5.4 Applicability of the Eurocode

Concerning the safety in a design, the most determining partial factor for hydraulic structures is the load partial factor for water level differences, which is defined as:

$$\gamma_w = \frac{u - B \cdot \log(\Phi(\alpha_s \cdot \beta_N) / f_N) - h_b}{u - B \cdot \log(F_{exd}) - h_b} \cdot e^{\{\alpha_R(\beta_N - 3{,}6)\cdot\sqrt{\ln(1+V_R^2)}\}} \quad (5.2)$$

However, this partial factor involves some issues. They are listed as follow:

- LK prescribes to use the load partial factor for water level difference on the water pressure. This results in incorrect values for the forces ($F \sim \Delta h^2$) and moments ($M \sim \Delta h^3$)
- LK prescribes the influences coefficients published by ISO 2394 (indicated in red in equation 5.2)
- The applied reliability Index (indicated in blue in equation 5.2). A discussion is created in which it is not clear which reliability index should be applied (Eurocode or LK)

In the following sections these issues are explained a bit more explicitly. The last two issues are processed into two case studies. Explained are the objectives of the case studies and how they contribute to a solution.

5.4.1 Partial factor for water pressure

In the LK the load partial factor is indicated as a partial factor for the water pressure caused by water level differences. This situation is sketched in Figure 5-2b. However, this water pressure is strictly taken incorrect. The truly acting water pressure is illustrated in Figure 5-2a. The water pressure is linear of the depth, and the acting force is a function of the squared depth. The water pressure marked in red from figure Figure 5-2c in the missing pressure (and force) in case the LK is applied.

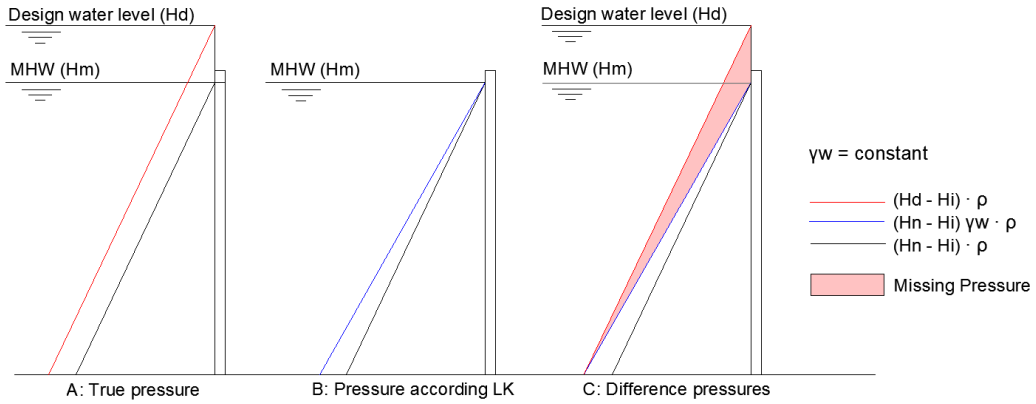


Figure 5-2: Water pressures conform LK and the truly acting water pressure

In practice this will not always lead to a wrong interpretation. Considering that the critical water pressure of both viewpoints is equal. For example, if in a calculation is calculated with the critical water pressure, this limitation is not included. See as an example the discussed case study in Chapter 5, in which the critical beam is fully loaded with the critical water pressure. If the door is vertically loaded by the water pressure the limitation is definitely noticeable. Further on in the report, this limitation will not be considered more specially in this research.

5.4.2 Applicability of the Influence coefficient (Case 2)

The influence coefficient indicates the extent of influence a certain elements has on the reliability of the structure. The Eurocode refers to the ISO 2394 standardized influence coefficients. These standardized coefficients make it accessible to define the partial factor for loads and strength parameters. A distinction between dominant load and strength parameters and non-dominant load and strength parameters is made. For dominant loads an influence coefficient of α = -0,7 should be used and for dominant strength parameters a coefficient of α = 0,8 should be maintained. For non-dominant parameter a coefficient of 0.4 multiplied with the influence coefficient for the dominant parameter (see also Table 5-1). In theory the sum of the influence coefficients in square should be equal to 1:

$$\sum \alpha_i^2 = 1 \tag{5.2}$$

The sum of the quadratic standardized influence coefficients of the ISO-method is bigger as one, which implies that the ISO-method is a conservative approximation.

Table 5-1: Standardized influence coefficients α

	Influence coefficient α
Dominant strength parameter	0,8
Non-dominant strength parameter	0,8 x 0,4 = 0.32
Dominant load parameter	- 0,7
Non-dominant load parameter	- 0,7 x 0,4 = - 0,28

These standardized coefficients are calibrated on basis of non-hydraulic loads. But in LK these same influence coefficients are used. In theory, these coefficients are not the correct values. They are based on wrong calibrations. This shortcoming will be researched in more detail in Section 7. In this section a probabilistic design is made for a lock gate. By adjusting the parameters the influence coefficient can be computed. In addition the recalculated influence coefficient can be entered into the determination method from equation 5.2. Subsequently, the determination method can be compared with the results from the probabilistic calculation.

5.4.3 Applicability of the reliability of the Eurocode in a Flood Defense System (Case 3)

A third interesting issue is the influence and applicability of the Eurocode on the flood defense system. The reliability classes in the Eurocode are based on the consequences and investments for building and bridges. By means of economic analyses, an optimum reliability can be found. For hydraulic structures (which are part of the flood defense system) this optimum reliability will not equal to those presented in the Eurocode, so based on that, it could be wrong to use the reliability from the Eurocode for hydraulic structures.

A lock should be seen as a part of the flood defense system, which is a whole different failure system as for buildings and bridges. Any changes in the reliability of the lock will eventually influence the failure probability of flooding of the area. This depends strongly of the composition of the failure system. The difference in failure system is illustrated in Figure 5-3. It indicates that there are more failure mechanisms present in the flood defense system with a much higher failure probability as is required

concerning the strength and stability. Eventually, the failure probability of the top event (flooding) will be much higher than the failure probability due to strength and stability. This means that also a change in the reliability of the strength and stability will have a limited effect on the reliability of the top event.

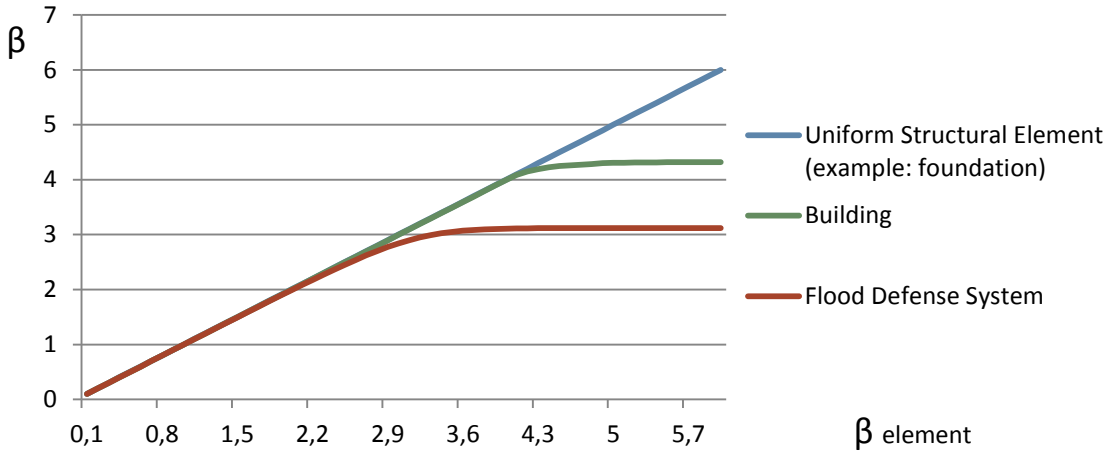


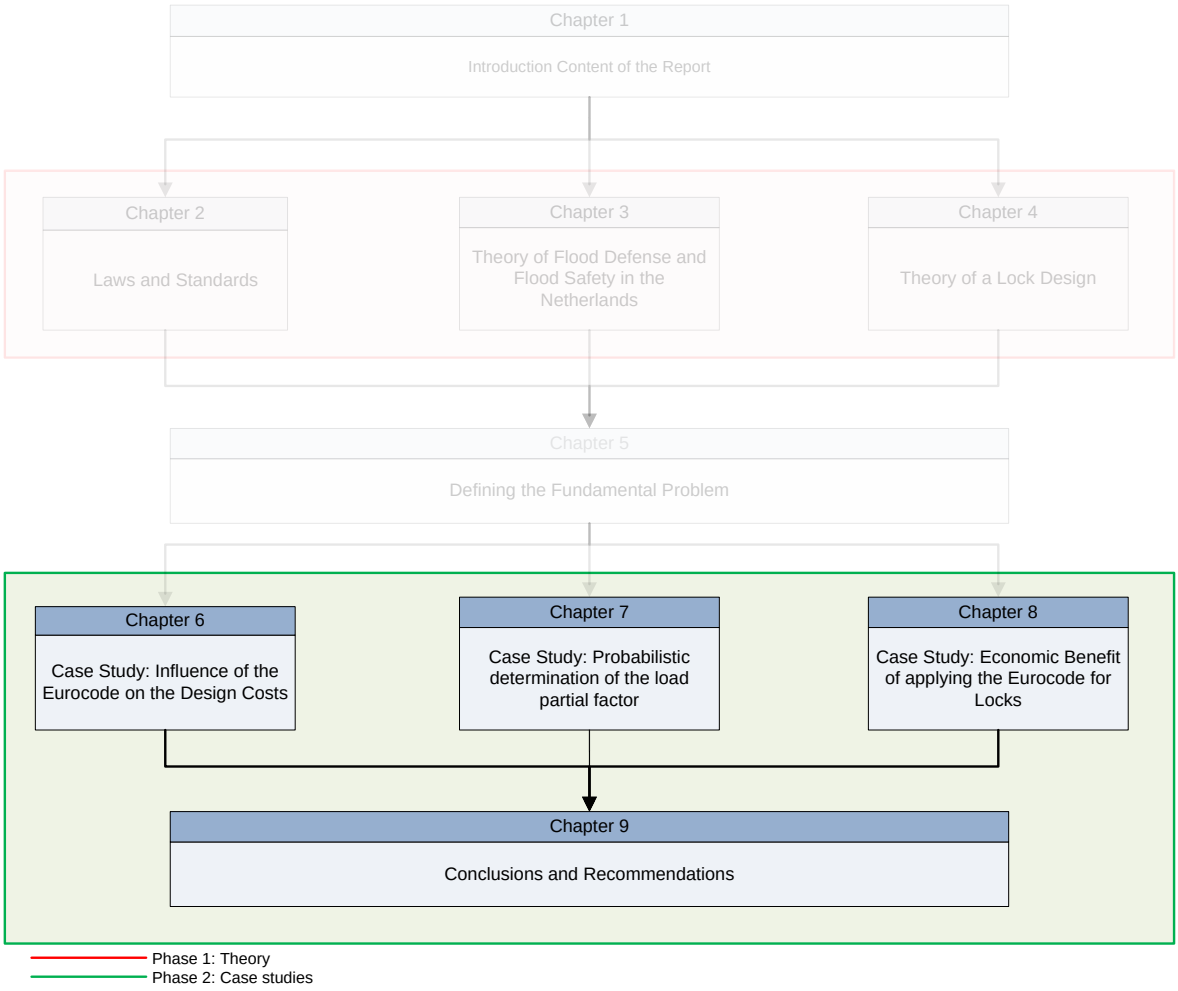
Figure 5-3: Example: Increasing the reliability of the foundation of a lock or a building, affects the failure system of which they are part differently. At some point, the increase does not affect the system failure probability anymore. This point is reached much sooner for locks than for building. This is causes due to the presence of failure mechanisms with a higher failure probability (such as overflow and closing)

Of course, it is difficult to say if the use of the Eurocode (so an increase of the reliability) is from economic point of view, an effective investment (profitable). It requires economic risk analyses to answer this question. The decrease in risk by applying the Eurocode and at the other hand, the investment will increase. The extent to which these two values changes decides if the investment is useful or not. Different parameters play a role in this consideration:

- Number of locks (N)
- Exceedance frequency of the dike-ring
- Failure probability of the dikes (Pf,dike)
- Consequence of flooding
- The required investment

In Section 8 the situation in which the Eurocode will be applied is discussed. A case study is carried out in which the sensitivity of the previous mentioned parameters is analyzed. The influence of the Eurocode on the failure probability of the flood defense system is considered and an indication is made of the influences on the risk analyses.

Phase 2


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Chapter 2
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Chapter 3
Theory of Flood Defense and Flood Safety in the Netherlands
Chapter 4
Theory of a Lock Design
Chapter 5
Defining the Fundamental Problem
Chapter 6
Case Study: Influence of the Eurocode on the Design Costs
Chapter 7
Case Study: Probabilistic determination of the load partial factor
Chapter 8
Case Study: Economic Benefit of applying the Eurocode for Locks
Chapter 9
Conclusions and Recommendations
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6 Case Study:

Influence of EC on the Costs

6.1 General

In the past subdivisions it is explained that the Eurocode has a different classification as the NEN-standards and the LK, which results in different reliability indexes. In the Eurocode a higher class is added. Hydraulic structures in the primary flood defense are structures with a high consequence by failure. BY definition hydraulic structures should be designed in the highest possible class. A higher reliability will lead to higher load and strength partial factors for the design. But what is the effect of using these factors in a design? Are the consequences noticeable or are they negligible? And what is the effect on the costs? In this chapter a case study is carried out in which these questioned are answered.

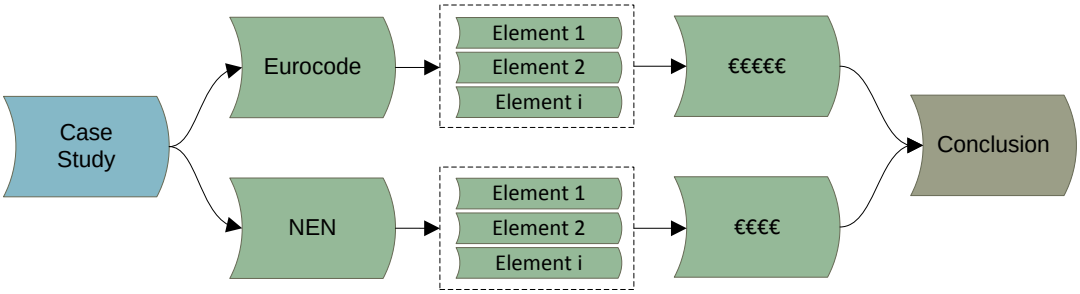


Figure 6-1: Structure of the case study

6.2 Elements

For the case study a navigation lock in the Netherlands is selected. In Section 4.3.1 the main components of a lock that would be relevant for the costs are defined. In this chapter the influence of the Eurocode is researched. The components which are discussed are:

- Lock gate (Section 6.5)
- Lock head (Section 6.6 and 6.8)
- Bed protection (piping) (Section 6.7)
- Lock chamber (Section 6.8)
- Lock approach (Section 6.8)

Each element will be designed on basis of a reliability class. The reliability class for an element is not fixed. So, for each element a different reliability class can be chosen. This results in a number of possible combinations. In Table 6-1 four combinations are defined. In the first combination, all the elements are designed according the NEN-standard. In the second combination, reliability class 2 of the Eurocode is applied and for the third combination reliability class 3 is applied. In the last combination, a combination of both class 2 and 3 are applied. This combination is explained in more detail in Section 4.4.3. In this combination there are three elements which should be designed on basis of the highest reliability class. Those are the elements which will possibly result to flooding. In this chapter these three elements are discussed explicitly (Chapter 6.5 to 6.7). The other elements are briefly discussed in Chapter 6.8, and more extensive in Appendix D and E.

Table 6-1: Combinations elements and reliability classes

	C1	***C2***	***C3***	***C4***	
	NEN (RC 3)	EC (RC 2)	EC (RC 3)	EC (RC 2)	EC (RC 3)
Lock gate	€	€	€		€
Lock head (strength)	€	€	€	€	
Lock head (stability)	€	€	€		€
Lock chamber	€	€	€	€	
Lock approach	€	€	€	€	
Bed protection	€	€	€		€
Total Costs	€	€	€	€	

In Table 6-2 the relevant standards for each element are mentioned, for the period before and after the Eurocode became valid. Remarkable is the limited influence of the LK on the elements of the lock. In fact, from this case study follows that only the gates are seriously affected by the LK.

Table 6-2: Replacement of the standards ordered by element

	Standard before 1th of April 2012		Standard after 1th of April 2012
Lock gate	LK & NEN	→	LK & EC
Lock head (strength)	NEN	→	EC
Lock head (stability)	LK & NEN	→	LK & EC
Lock chamber	NEN	→	EC
Lock approach	NEN	→	EC
Bed protection	NEN	→	EC

6.3 Introduction Case

The lock that is selected for the case study is the new navigation lock in Eefde, a small place in the Netherlands. In the Figure 6-2 the current situations is shown. The complex consists of a dewatering gate (south) and a navigation lock (north). This navigation lock was recently in the news, because one of the lift gates felt down. The incident was caused by the failure of a threaded rod. The main reason why this navigation lock is chosen is because of the accessibility of information.

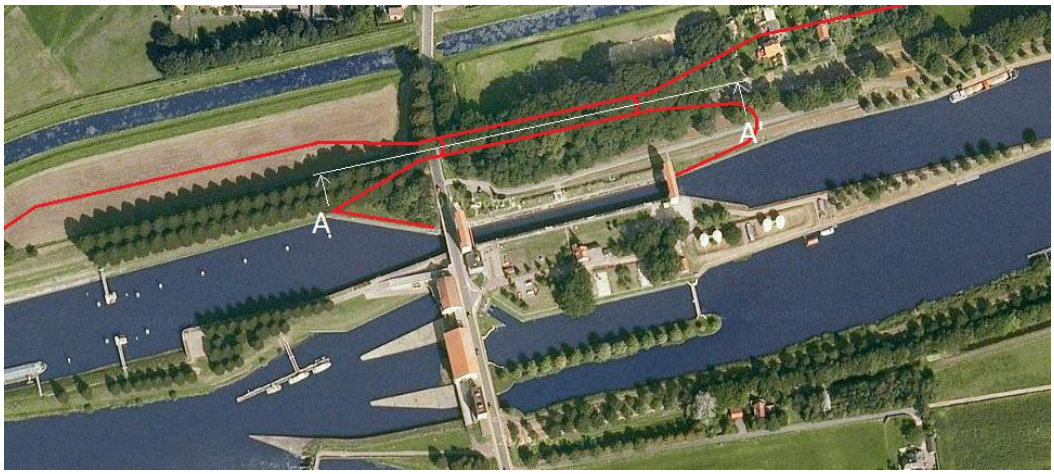


Figure 6-2: Location of the new navigation lock in Eefde

Rijkswaterstaat (public authority) approached Grontmij for research of the possibilities to expand the lock complex, which is required because currently the waiting times are too long. The requirement is a maximum waiting time of 30 minutes. For quite some time this requirement is exceeded. A better exploitation of the current lock will not solve the problem. For that reason a second lock will be constructed next to the current lock. After a feasibility study the location is set (expressed with the red lines in Figure 6-2) and a design is made (by Grontmij). In Figure 6-3an overview of the lock is illustrated. All the elements that are considered are numbered and briefly described beneath the figure.

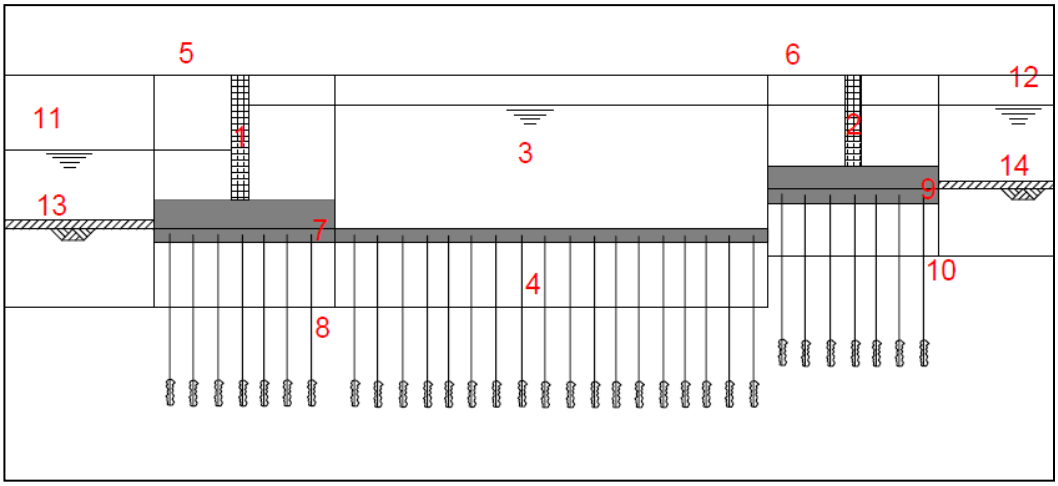


Figure 6-3: Cross-section (A – A' from Figure 6.2) of the design of lock Eefde

1 and 2:

These two elements are the lower (1) and upper (2) gate. The upper gate is in connection with the IJssel and the lower gate with Twentekanalen. Each gate contains one set of gates. For both gates the mitre type is used and the material is steel.

3 and 4:
These elements indicate the lock chamber. It consists of two sub elements: the combi-wall (3) and the underwater concrete floor (4). The concrete floor in founded on a series of grout piles.

5 and 6:
The lower (5) and upper (6) concrete lock head both are an U shaped concrete box.
The upper lock head is founded on a higher level.

7 and 8:
These elements indicate the construction pit of the lower lock head. Number 7 points out the underwater concrete floor. The floor is founded on a series of grout piles. As wall of the construction pit a combi-wall is used. After the lock head is placed, the upper part of the wall will be cut off for reuse (from the top of the lock head till the bottom of the lock head). The lower part of the combi-wall will function as cut-off screen, to increase the piping length.

9 and 10:
These elements stand for the construction pit of the upper lock head. See 7 and 8 for further explanation.

11 and 12:
The lock approach at IJssel (11) and at the channel (12) indicates the navigation area between the connecting waterway and the lock complex, where approaching vessels have the opportunity to decrease speed and moor to a guiding structure if necessary. A lot of combi-walls should be placed to meet the required capacity.

13 and 14:
With these numbers the bed protection is pointed out. For these area a layer different stone sizes are placed which block the sand from flowing away. Finally a layer of colloidal concrete is deposited

6.4 Hydraulic boundary conditions

The boundary conditions for the water levels are given in Table 6-3. Remarkable is that the water level in the channel is higher than in the IJssel. This is unusual for primary flood defenses. Normally a flood defense will protect the inland from inflowing water. But in this case the water level in the inland is always higher. Theoretically it could happen that a dike in the channel will collapse, which results in a lower water level in the channel. In that case the water level in the IJssel could be higher. But that scenario is not included in the design and research. Rijkswaterstaat has reported that Grontmij should consider the navigation lock in Eefde as a part of the primary flood defense, which means it should meet the standards according to the Waterwet.

Table 6-3: Critical water levels in the IJssel and channel

Load	Value	Frequency
Ijssel		
MHW	NAP + 8.80 m	1/1250
Mean water level	NAP + 4.30 m	
OLR	NAP + 2.46 m	
OLR (1 year)	NAP + 1.46 m	
MLW	NAP + 1,07 m	1/1250
Twentekanaal		
Mean water level	NAP + 10.00 m	
Low canal level	NAP + 9.870 m	1%
Min low canal level	NAP + 9.870 m	
High canal level	NAP +10.140 m	1%
Max high canal level	NAP +10.150 m	

In active state the highest water level differences are shown in Figure 6-4. On IJssel the minimum water level is +1.46 meter, the so called OLR (overeengekomen laagste rivierstand). If the water level is lower than the OLR, ships will not be able to sail into the lock chamber. For the upper gate the critical situation occurs for a maximum water level at the channel side and a water level lower as +5,65 on the IJssel. Since the mean water level in the IJssel is +4.30 NAP, this can happen quite often.

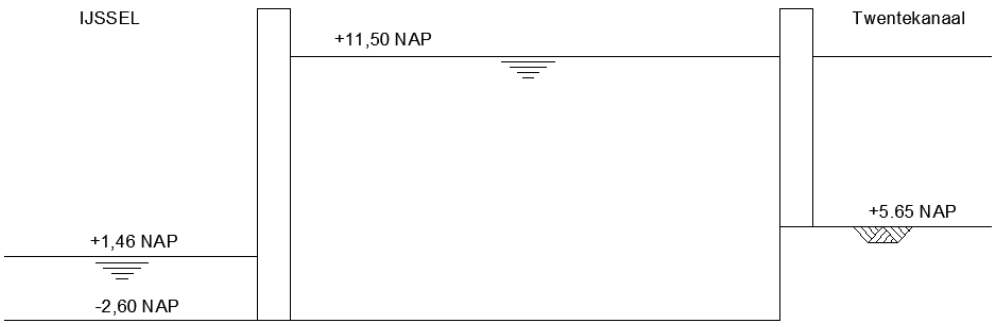


Figure 6-4: Critical water levels lower gate

6.5 Lock Gate

The lock in Eefde is just locking one side, the channel side. On each side there is one set of gates installed, from which follows a total amount of 4 gates. The gates of the upper and lower gate differ in design. The design of the lower gate is shown in Figure 6-5. The material from which it is constructed is steel. Seven steel profiles (H-profiles) are connected with each other by a steel plate. On the sides two steel profiles are placed.

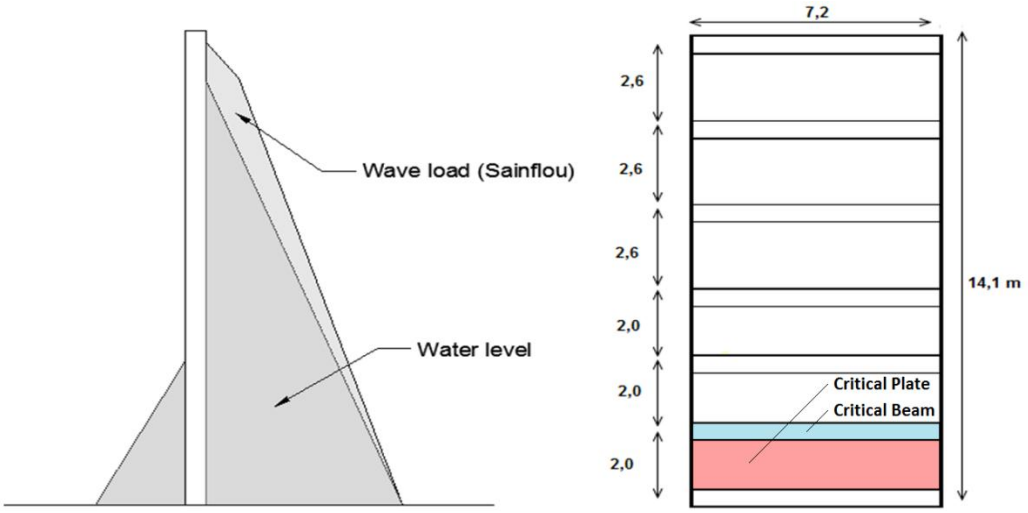


Figure 6-5: Dimensions of the lower gate and relevant loads

6.5.1 Loads

The gate is dealing with several loads. In the Figure 6-5 the loads are expressed as pressure on the gate. From there on the critical profile can be found, which will be the second from the bottom. The details of this calculation can be found in Appendix C. The loads that are taken into account are:

- Maximum water level differences
- Wave loads
- Wind loads
- Translation waves

In Table 6-4 the relevant partial factors are shown that will be used for the design of the gate. These partial factors are used on resulting forces of the loads. The marked partial factors are defined in the LK (see 3.5.3).

Table 6-4: Partial factors which are used for the design of the gate

Loads	NEN (RC 3)	EU (RC 2)	EU (RC 3)
- Water levels	1.25*	1.30*	1.50*
- Wind/Wave loads	1.25*	1.30*	1.50*
- Translation waves	1.50	1.50	1.65
*These partial factors are derived from the method as is explained in the LK (see 3.6)			

6.5.2 Calculation

For the calculation two items are checked. The first item is already mentioned, the critical beam (marked in blue in Figure 6-5), and second the thickness of the front plate (marked in red in Figure 6-5). In practice, between the beams orthotropic plates are placed to increase the stiffness of the gates. If those plates are used, the thickness of the front plates can be reduced. As a simplification for the calculations, the orthotropic plates are not included

Verification method (steel)

For the calculation of the beam the bending moment will be dominant. Both standards will influence the design by the verification method and the partial load factors. From the calculations (which can be found in Appendix E) it can be noticed that with regard to the verification of steel beams for bending moments, there is not a large difference between the NEN and the Eurocode. Figure 6-6 illustrates for both standards the unity check for bending moment as a function of the partial load factor (water level difference). In the same Figure it is also shown what the influence is of the partial factor.

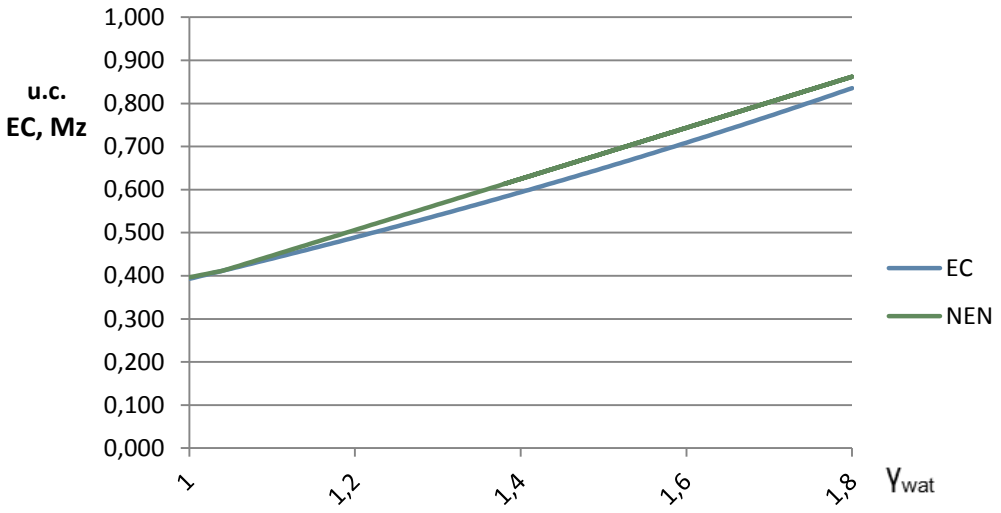


Figure 6-6: Result (in u.c.) of the different verification method between NEN and EC

Beam

Regarding the partial load factors, a large influence is causes by the partial factor for water level differences. For this reason the unity checks of the beam will be expressed as a relation of the partial factor for water level differences. For the strength the elastic bending moment is taken into account and the unity check for bending moment of the critical beam can be defined as follow:

$$u.c. = \frac{\frac{1}{8} \cdot (h_{up} - h_{low}) \cdot \gamma_w \cdot x_{h.o.h.} \cdot L^2 \cdot \rho_{d,w} \cdot g}{f_{y,sp} \cdot W_{y,el}} \tag{6.1}$$

In which:

h_{up} = water level of the high water side on the critical beam [m NAP]
h_{low} = water level of the low water side on the critical beam [m NAP]
γ_w = partial factor for water level differences [-]

$x_{h.o.h.}$ = centre to centre distance between the beams [m]
L = length of the beam [m]
$\rho_{d,w}$ = water density [kg/m^3]
$f_{y,sp}$ = strength of the steel [MPa]
$W_{y,el}$ = plastic section modulus [m^3]
g = gravitation [m/s^2]

In Figure 6-6 the influence of the partial factor for water level differences is illustrated for several types of beams. In the figure the points of intersection with the different types of standards are marked. It can be easily noticed what the effect is on the unity check by applying a different standard (on a specific beam).

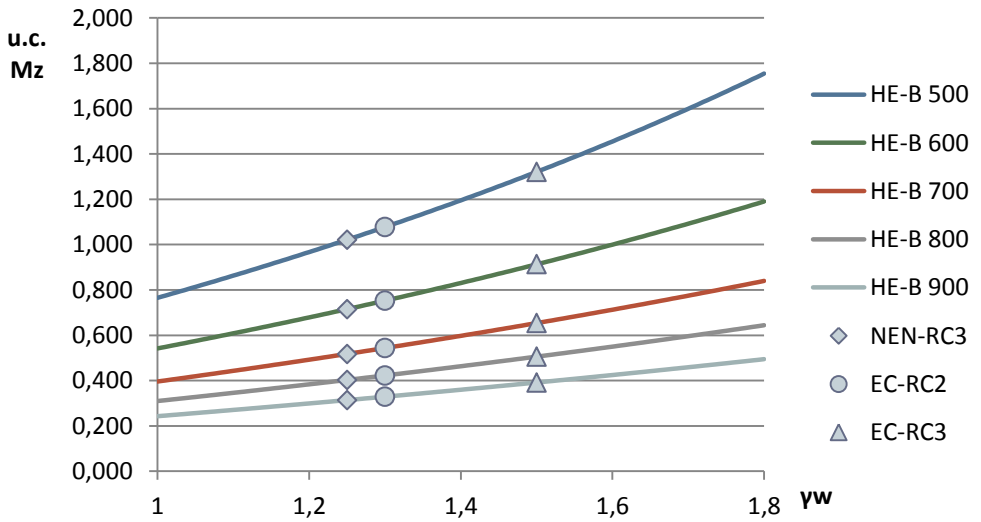


Figure 6-7: Relation u.c. bending moment with partial factor yw (yw = water level difference)

Front Plate

The second element that should be checked is the front plate. The plate is critical loaded for bending moment. The unity check for the front plate is defined as:

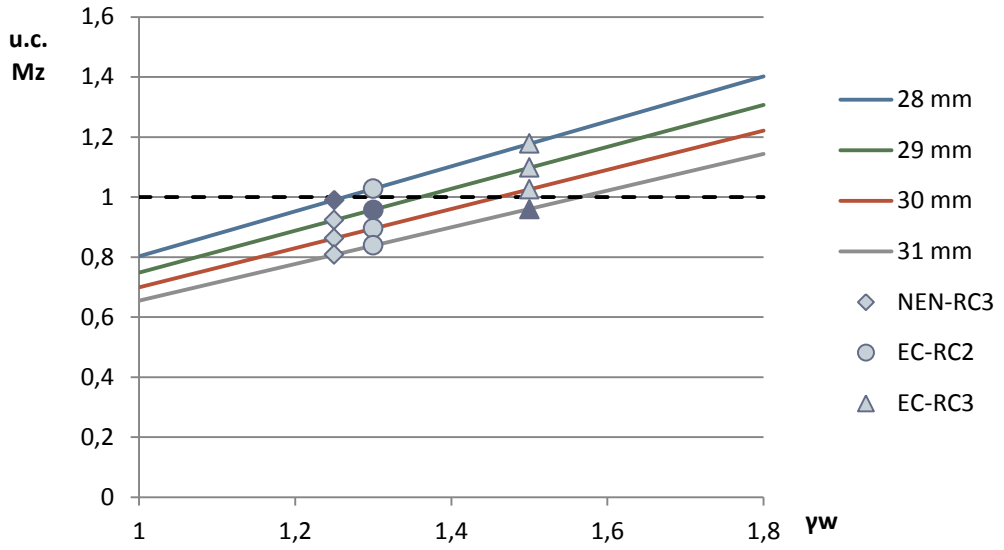
$$u.c. = \frac{\frac{1}{10} \cdot (h_{up} - h_{low}) \cdot \gamma_w \cdot {x_{h.o.h.}}^2 \cdot d \cdot \rho_{d,w} \cdot g}{f_{y,sp} \cdot W_{y,el}} \qquad (6.2)$$

In which:

h_{up} = water level of the high water side on the critical plate area [m NAP]
h_{low} = water level of the low water side on the critical plate area [m NAP]
γ_w = partial factor for water level differences [-]
$x_{h.o.h.}$ = length of the plate (equal the c.t.c. distance between the beams) [m]
$\rho_{d,w}$ = water density [kg/m^3]
$f_{y,sp}$ = strength of the steel [MPa]
$W_{y,el}$ = elastic section modulus (1/6 * b * d^2) [m^3]
d = thickness of the plate [m]
g = gravitation [m/s^2]

In strength the unity check is influenced by the thickness of the plate. A higher thickness results in a higher section modulus. In Figure 6-8 the relation of the partial factor of water level differences and the unity check is expressed for four different thicknesses. The points of intersection with the different types of standards are marked in the figure. With this figure it makes it easy to decide which thickness for which standard should be used.

Figure 6-8: Relation thickness plate with partial factor yw (yw = water level difference)



Design decisions

With the analysis of the critical beam and front plate, a decision can be made about the dimensions of the elements. The decisions are listed in Table 6-5 for each standard and reliability class. Regarding the beam, only for the highest class in the Eurocode a HE-B700 profile will be used. In the two other classes a lower profile will be sufficient. This decision follows from Figure 6-7. It is chosen to design the structure with an unity check of maximum 0,9 which explains the different profile between the classes. For the front plate a small difference in thickness can be found.

Table 6-5: Decisions in the design

	NEN -RC3	EC-RC2	EC-RC3
Beams	HE-B600	HE-B600	HE-B700
Front Plate	28 mm	29 mm	31 mm

It should be noticed that the dimensions are strongly depended of the person who is designing, because they are based on decisions. The results from the calculations are unity checks. Strictly taken a unity check of 0.99 is a safe design. But in practice there are not many engineers who design a structure based on such a unity check. It is all about the choice of the engineer. It also often happens that the size of the beam is determined by the construction method (for example mounting and welding) or by standardized sizes (in case of plate thicknesses).

6.5.3 Costs

From the design the required amount of material can be calculated. From the amount of material an estimation of the costs can be made. At Grontmij the program GWW-Calc is used for estimations like these. In Figure 6-9 an overview of the costs of the lower gate is expressed. It concerns the costs made during the lifetime. In Appendix F a more explicit overview pictured.

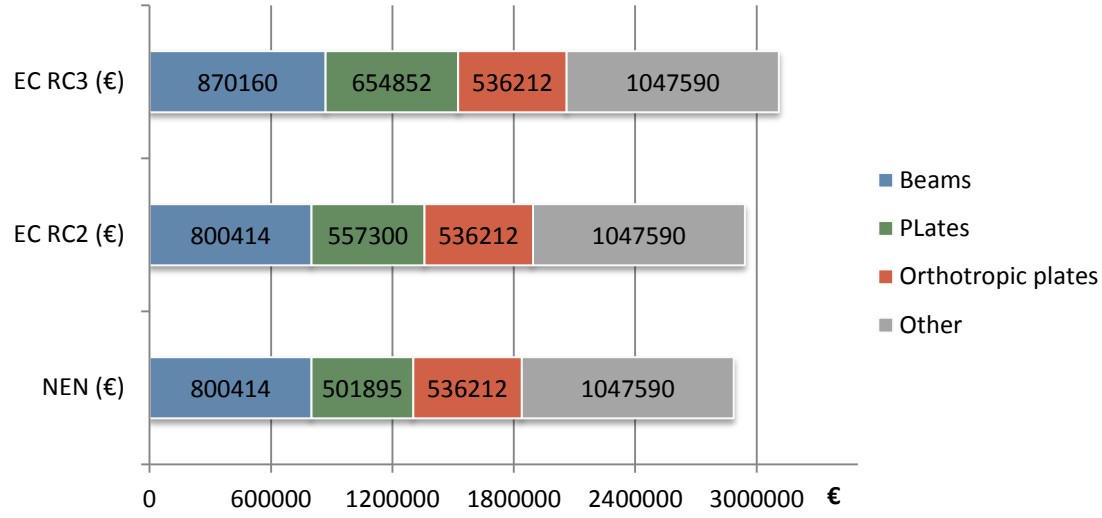


Figure 6-9: Overview of the costs of the lower lock gate

The overall differences between the standards are approximately 200.000 euro, which is 7 % of total costs of the lower gate. For the upper gate the differences are a bit smaller (3%). Relatively to cost of the whole project, this difference is very small. But it is still a number that should not be neglected. If the values are compared with the total project cost (about 90 million) is results in an increase of **0,32%** (both gates).

Tabel 6-6: Total cost for the upper and lower gate

	NEN RC3 [€]	EC RC2 [€]	EC RC3 [€]
Upper Lock Gate	1.510.338	1.510.338	1.599.845
Lower Lock Gate	2.886.111	2.941.517	3.108.815
Total	4.396.449	4.451.854	4.708.659
Difference	-	55.405	312.210

6.6 Stability Lock Head

One of the failure mechanisms that will lead to flooding is failure of the stability. For stability the lock head, or any other structure, is considered as a monolithically thing on a foundation. Checking the stability of the structure translates into checking the strength of the soil under the structure. There are three type of instability:

- Vertical stability (Heave)
- Horizontal stability
- Overturning

Vertical stability

Vertical instability will occur if the uplifting water pressure under the lock head is too high (heave). This failure mechanism is already done for the underwater concrete floor of the construction pit and satisfied the standard. If the lock head will be placed on top of the floor the force downward will be higher, which has a positive effect on the verification for heave. For this check the partial factors in Table 6-7 are relevant. There is no distinction made in class. In the Eurocode as well as in the former NEN, the same factors are used, which will lead to no differences between the two standards at all.

Table 6-7: Relevant partial factors concerning heave

	Permanent Unfavorable	Favorable	Variable Unfavorable
NEN	1.1	0.9	1.5
Eurocode	1.1	0.9	1.5

Horizontal stability

The verification for the horizontal stability is carried out in appendix E. The forces that are included in the verification are expressed in Figure 6-10. Before the lock head is placed, an underwater concrete floor has been installed. For the horizontal stability no contribution of the grout anchors has been taken into account. At the time that the anchors are contributing to the vertical stability, the displacement is already too big. The friction between the sand and the bottom of the lock head should take the resulting horizontal forces.

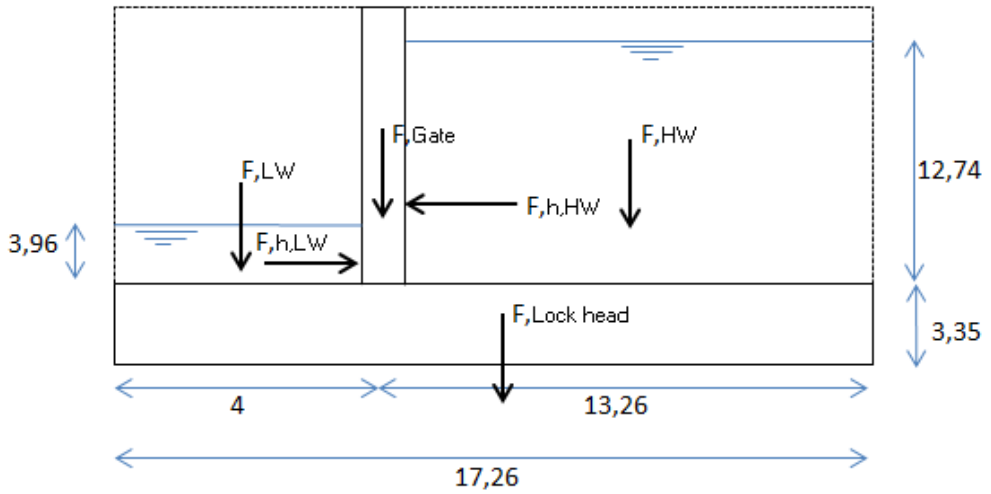


Figure 6-10: Cross-section of the lower lock head and the relevant forces

For the horizotal stability the following equilibrium is set:

$$\sum H < f \cdot \sum V \tag{6.3}$$

In which:
- ∑H = Resultant force of the horizontal water pressure on the gates and the lock head; [kN]
- ∑V = Resultant force of the vertical forces, being: the dead weight of the lock head and gate(s), the weight machinery or equipment, the weight of the water above the lock floor; reduced by the upward (groundwater) pressure; [kN]
- f = tan(2/3 ϕ) [-]
- ϕ = Angle of internal friction of the subsoil [º]

The partial factors that are used for the vertical equilibrium is presented in Table 6-8.

Table 6-8: Relevant partial factors concerning vertical stability and overturning

	Permanent Water	Permanent Unfavorable	Favorable	Variable Unfavorable
NEN (RC3)	1.25	1.1	0.9	1.5
Eurocode (RC2)	1.30	1.1	0.9	1.5
Eurocode (RC3)	1.50	1.1	0.9	1.5

Overturning

After checking the vertical and horizontal equilibrium, the structure has to be checked on overturning. For overturning the grout piles do play a role in the equilibrium. The full calculation for overturning can be found in Appendix E. The equilibrium that should be met is:'

$$\frac{\sum M}{\sum V} < \frac{1}{6} b \tag{6.4}$$

In which:
- ∑M = Overturning moment will be computed using the arm between Bottom of Structure (BoS) and the working line of the horizontal resultant, and the arm or eccentricities of the vertical force(s); [kNm]
- ∑V = Resultant force of the vertical forces, being: the dead weight of the lock head and gate(s), the weight machinery or equipment, the weight of the water above the lock floor; reduced by the upward (groundwater) pressure; [kN]
- b = Length of the lock head. [m]

The ratio between the moments and the vertical load indicates an eccentricity of the vertical component of the resulting force. This requirement could be popularly phrased as: 'No tension allowed' under the base slab of the structure'. To meet this requirement, the vertical component of the resulting force has to stay within the core of the foundation, a distance which has to be smaller than 1/6 of the base.

Results

In Table 6-9 the unity check each failure mechanism is expressed. As already said, for heave there are no differences between the standard. For the other two mechanisms small changes can be seen in the unity check. These changes are mainly caused by the increase of the partial factor of water levels.

Table 6-9: Results of the stability of the structure

	Horizontal	Vertical	Rotation
NEN (RC3)	0,38	0,48	0,32
EC (RC2)	0,38	0,54	0,34
EC (RC3)	0,38	0,71	0,43

Table 6-9 shows that influences are considerable; however the unity checks are close to each other. In this case study, the influence concerning stability is not taken into account in the total costs. This could be explained as follow; the process of designing, start at a rough level and ends in a detailed level. The verification of stability is normally made in an early stage of the design. In this stage, a lot of decisions are still open to consider the dimensions of the lock. In this context, the verification concerning stability has much influence on the dimensions of the lock. Unfortunately, in the considered case study, a design was offered from Grontmij. This design was in much further stage, in which changing the dimensions was not convenient any more. For this reason, the dimensions are unchanged, in which the influence of the verification on stability is neglected in the total cost.

6.7 Piping

Piping is the process of (ground) water flow, in the soil under and/or around the structure, having enough transport capacity to erode soil particles. As soon as the first particles are carried away, more water will be able to flow along the same route because of the reduced flow resistance and more soil particles will be washed away. The process strengthens itself and a pipe will develop. This will undermine the structure and, in the end, result in instability of the whole structure.

Piping can be avoided by extending the pipe length. Possibilities of doing this are bed protection or cut-off screens. Bed protection ensures that the out flowing water cannot take any particles with it. Until the rate of the outflow is under the critical flow, bed protection is required. The cut-off screens are installed under the structure and extend the pipe length. In this case study, both methods are used. In Figure 6-11 an overview of the lock is shown and the locations of the bed protection are given.

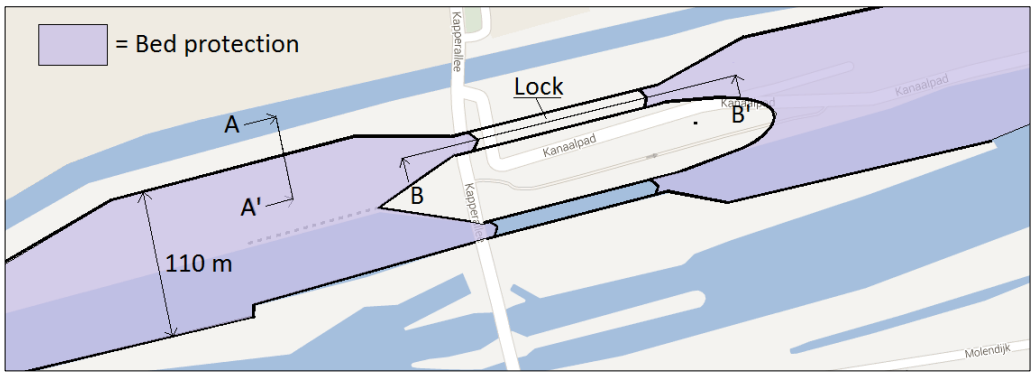


Figure 6-11: Map of the used bed protection and cross sections

There are two critical locations which should be analyzed for failure due to piping. These locations are indicated in Figure 6-11 with the cross section. Both locations should suffice the minimal required pipe length. This is done by calculating the minimal required pipe length, which can be calculated by the rule of Lane:

$$\Delta H \le \Delta H_c = \frac{\left(\frac{1}{3} L_h + L_v\right)}{\gamma_{piping} \cdot C_{w,piping}} \tag{6.5}$$

In which:

ΔH	= Water level difference		[m]
ΔH_c	= Maximum allowed water level difference		[m]
L_h	= Horizontal seepage length		[m]
L_v	= Vertical seepage length		[m]
$C_{w,piping}$	= Seepage-factor = 6 (for course sand)		[-]
γ_{piping}	= Partial factor:	NEN ➔ 1,5	
		Eurocode ➔ 2,0	

6.7.1 Quay wall

The quay wall is sketched in Figure 6-12. A flow is presence from the inland to the channel water and can be prevented by impermeable walls or a bed protection.

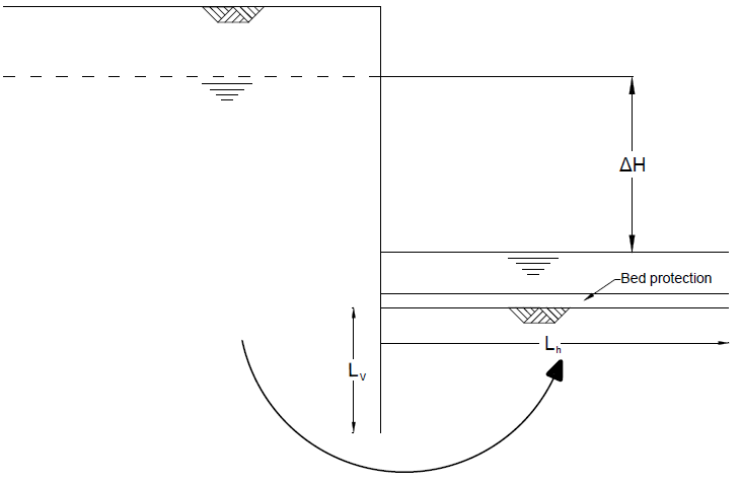


Figure 6-12: Piping in cross direction at the combi-wall (cross section A-A')

The situation sketched in Figure 6-12 is a conservative approach. It is assumed that the groundwater level is +7 NAP and the water level equal to +1,46 NAP. The fluctuation of the water level in the IJssel (on the location of the lock) is very slow (period of 20 days). This means the groundwater level has enough time to adjust to the open water. This occurrence will be neglected. The required seepage length is found as follow:

ΔH = Groundwater – low water = (+7 NAP) – (+1,46 NAP) = 5,54 NAP

L_h = This will be the length of the required bed protection over the width of the channel

L_v = The length of the sheet pile is 4,2 m.

From the calculation of the Rule of Lane, it follows that there is a significant difference between the minimum required pipe length (with that the bed protection) regarding both standards. But it is also observed that the required length of the bed protection (for both standards) is much larger than the width of the channel. Therefore the full width of the channel should be protected against piping. Concerning this location, there is no difference visible in the design regarding the bed protection. The quay is about 250 meter long at each side of the structure (total 500 meter). This means that there is a bed protection required of 55000 m^2.

Table 6-10: Calculation results regarding cross sectional piping

	NEN	Eurocode
γ_{piping}	1,5	2,0
Minimum L_h	137 m	187 m
Width channel	110 m	110 m
Length quay	500 m	500 m
Surface bed protection	55000 m^2	55000 m^2

6.7.2 Under and along the structure

The second situation is sketched in Figure 6-13. Normally a structure should be verified for piping under and along the structure. However, the structure is connected with the quay wall. This wall has approximately the same length as the cut-off screens from the construction pit. In anyway, the water should go under these walls to pass the structure. Thus, both piping under and piping along results in the same pipe length.

In Figure 6-13 the length L_h represents the length of the bed protection. The bed protection is as long as the combi-wall of the lock approach extends. This is around 250 meters at each side (total 500m). The L_v represents the length of the cut-off walls. These walls are the remained combi-walls from the construction pit. The upper part of the walls of the construction pit is cut off for reuse, and the lower part stayed in the ground to prevent piping. For the combi-wall only the depth of the sheet pile will be relevant (and not the depth of the tubular pile).

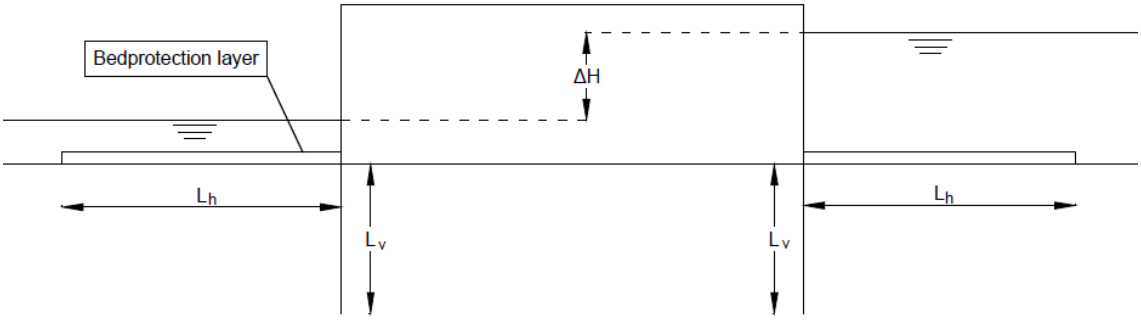


Figure 6-13: Piping in longitudinal direction concerning the navigation lock (cross section B-B')

ΔH = High water – Low water = (+10,14 NAP) – (+1,46 NAP) = 8,68 NAP

L_v = The depth of the sheet pile at the lower side is 1.1m and at the upper side 2.85. This makes the total vertical creep length 7.9m.

L_h = This will be the sum of the length of the bed protections on both sides. The length under the lock should not be taken into account, because it is founded on a pile foundation.

Table 6-11: Calculation results regarding piping in longitudinal direction

	NEN	Eurocode
γ_{piping}	1,5	2,0
Minimum L_h	211 m	289 m
Length bed protection	500 m	500 m

There is a large difference in minimal required pipe length. But in this situation the piping mechanism in cross direction is critical (at the quay wall). It demands that a bed protection is required over the full length of the lock approach, which is in total 500m. The pipe minimal length which follows from piping in longitudinal direction (under and along the structure) is much lower. So, eventually there is no difference visible (between the Eurocode and NEN) in the design regarding the bed protection.

6.8 Other Elements

In Appendix D an explicit calculation can be found of the other elements which are relevant for the design of a lock. In short the main issues of each element will be discussed in the following section.

Lock head (strength)

The calculation for the lock head is carried out with the program SCIA Engineer. With this program the critical forces and stresses can be found. From the calculations regarding to the bending moments, in most sections the minimal reinforcement will be sufficient. As a consequence of the Eurocode small differences in reinforcement rate does occur.

If the cross section will be checked for shear forces a problem arises. The interpretation of verification on shear stress between the standards (NEN and EC) is quite different. In Figure 6-14 the capacity and the present shear stress is expressed as a function of the thickness. From this figure it is easily to find the required thickness for a certain shear stress. In addition, the minimal shear stress value conform the NEN is expressed. From there the differences between the NEN and the Eurocode can be easily found. It can be noticed that there is a large gap between the required thickness for NEN and Eurocode. This gap can be closed by applying stirrup reinforcement that is also expressed in the figure.

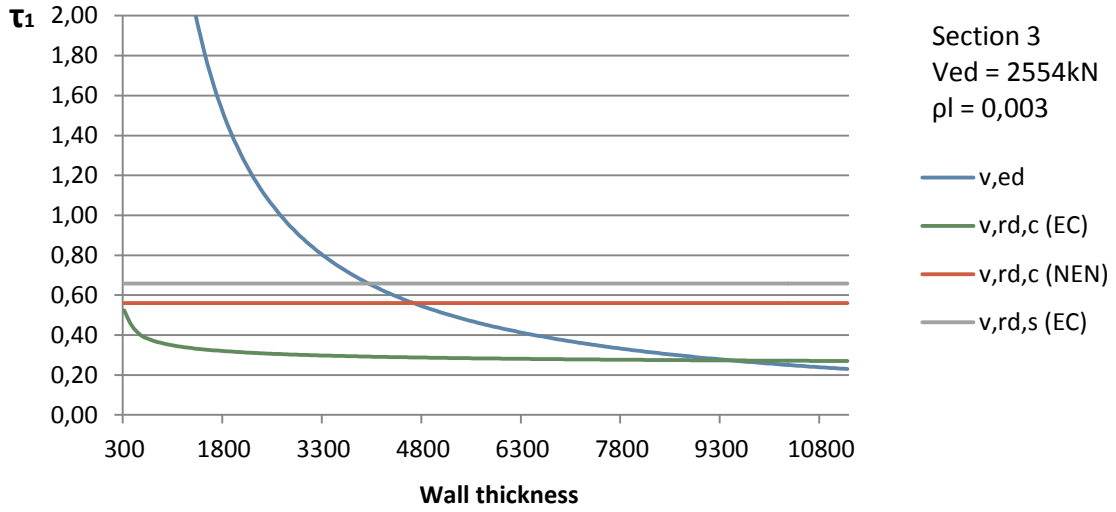


Figure 6-14: Relation capacity, present and reinforcement shear stress as a function of the thickness

Construction pit

The calculation of the construction pit is done with the program D-sheet. Recently the Eurocode is added in the program. So the calculation can be easily performed for both standards. The results of the calculations are almost identical. Only the minimal required length is not equal in both standards. The identical results can be explained by the steps of verification conform CUR 166. In general, sheet piling design involves the analysis of all relevant Ultimate Limit States (Failure) and Serviceability Limit States during each stage of construction. In Table 6-12 all the combinations are given in combination with the partial factor that is used. For the design of the construction pit the last step will be leading (SLS), which explains the similar results.

Table 6-12: Verification steps according to CUR. Due to the fact that the last step is critical, no distinctions are found between the standards. For other calculations this could be different.

Step	Limit State	k_d	c_d	$\tan(\phi_{d})$	$\tan(\delta_d)$
6.1	ULS	$k_{low;rep} / \gamma_k$	$c_{low;rep} / \gamma_c$	$\tan(\phi_{low;rep}) / \gamma_{\tan(\phi)}$	$\tan(\delta_{low;rep}) / \gamma_{\tan(\delta)}$
6.2	ULS	$k_{high;rep} / 1.0$	$c_{low;rep} / \gamma_c$	$\tan(\phi_{low;rep}) / \gamma_{\tan(\phi)}$	$\tan(\delta_{low;rep}) / \gamma_{\tan(\delta)}$
6.3	ULS	$k_{low;rep} / \gamma_k$	$c_{low;rep} / \gamma_c$	$\tan(\phi_{low;rep}) / \gamma_{\tan(\phi)}$	$\tan(\delta_{low;rep}) / \gamma_{\tan(\delta)}$
6.4	ULS	$k_{high;rep} / 1.0$	$c_{low;rep} / \gamma_c$	$\tan(\phi_{low;rep}) / \gamma_{\tan(\phi)}$	$\tan(\delta_{low;rep}) / \gamma_{\tan(\delta)}$
6.5	SLS	$k_{low;rep}$	$c_{low;rep}$	$\tan(\phi_{low;rep})$	$\tan(\delta_{low;rep})$
6.5*1.2	ULS	$\mathbf{k_{low;rep}}$	$\mathbf{c_{low;rep}}$	$\mathbf{\tan(\phi_{low;rep})}$	$\mathbf{\tan(\delta_{low;rep})}$

The difference in minimum lengths of the sheet piles is not large. The main reason for this is a new requirement in the Eurocode. The length of the retaining wall should be extended with 10%, with a maximum of 0.5 meter.

The Eurocode will not directly lead the differences for the dimensions of concrete floor, because the critical elements will not be influenced. The floor and piles are verified on the following failure mechanisms: capacity of the piles, heave and strength (internal stress)

Lock chamber and lock approach

For the lock chamber and the lock approach the same results are found as for the construction pit. No large differences are found in the earth retaining walls. Only the minimal required lengths of the walls differ in small amounts.

6.9 Results on the Costs

After analyzing all the elements a conclusion can be made about the influence of the Eurocode on a design of a lock. It must be said that each lock is different, which will eventually results in different quantities of the influences. The total project costs can be calculated by adding up all the separate costs for each element. In previous sections the costs regarding mechanics and electrics are not mentioned. These costs are taken from the original estimation and are assumed to be same for both standards. The costs of the project are lined up in Table 6-13. From this table a comparison can be made between the standards.

Table 6-13: Project costs according to different standards

	NEN RC3 (CUR II)		NEN RC3 (CUR III)		EC RC2		EC RC3	
Upper gate	1.510.338	€	1.510.338	€	1.510.338	€	1.599.845	€
Lower gate	2.886.111	€	2.886.111	€	2.941.517	€	3.108.815	€
Construction pit Channel	3.481.820	€	3.540.577	€	3.523.789	€	3.557.365	€
Construction pit IJssel	3.120.513	€	3.170.152	€	3.155.969	€	3.184.334	€
Foundation	2.533.021	€	2.533.021	€	2.533.021	€	2.533.021	€
Upper structures	11.723.020	€	11.723.020	€	11.723.020	€	11.723.020	€
Lock head Channel	4.326.533	€	4.326.533	€	4.434.879	€	4.495.000	€
Lock head IJssel	3.290.789	€	3.290.789	€	3.466.120	€	3.562.635	€
Lock Chamber	6.739.125	€	6.802.353	€	6.802.353	€	6.827.644	€
Lock approach Channel	2.180.880	€	2.221.361	€	2.221.361	€	2.232.927	€
Lock approach IJssel	25.709.954	€	26.187.174	€	26.187.174	€	26.323.523	€
Mechanics/Electrics	28.068.704	€	28.068.704	€	28.068.704	€	28.068.704	€
Bottom protection	2.198.381	€	2.198.381	€	2.198.381	€	2.198.381	€
Total	97.769.188	€	98.458.513	€	98.766.625	€	99.415.212	€
	-		+ 0.68%		+ 1.00%		+ 1.67%	

Some interesting influences can be observed from the case study. In the table the first column represents the costs for a design based on the NEN and CUR II. This combination is taken as the representative costs. Figure 6-15 shows the additional costs for the other standards as a percentage of the representative costs (distinguished by element). Obviously the lock approach, lock head and lock gate contribute considerably to the additional costs.

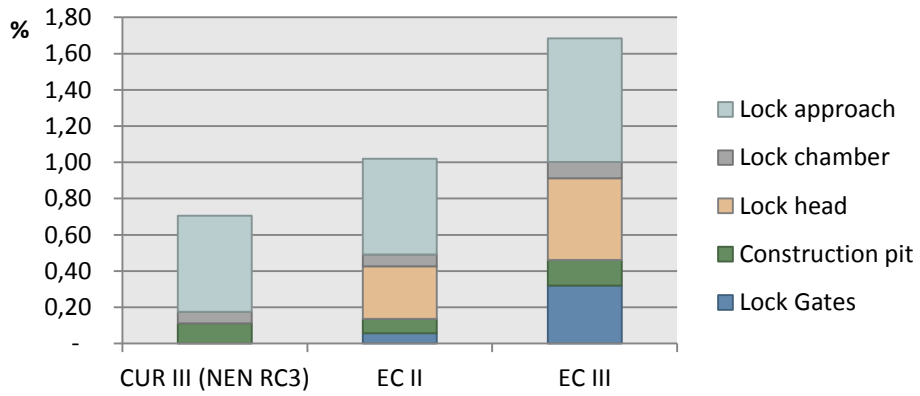


Figure 6-15: Comparison of the standards (on overall costs) with the NEN in combination with CUR II (Cumulative)

In Figure 6-16 the situation is shown in which the NEN-standard in combination with the CUR III is set as representative. It shows the influences of reliability class 2 and 3 for each element. It is observed that the lock head and the lock gate have a high contribution to the additional costs. The lock approach disappears from this comparison, which is logic considering the fact that the reliability of the CUR III and the EC are not far from apart.

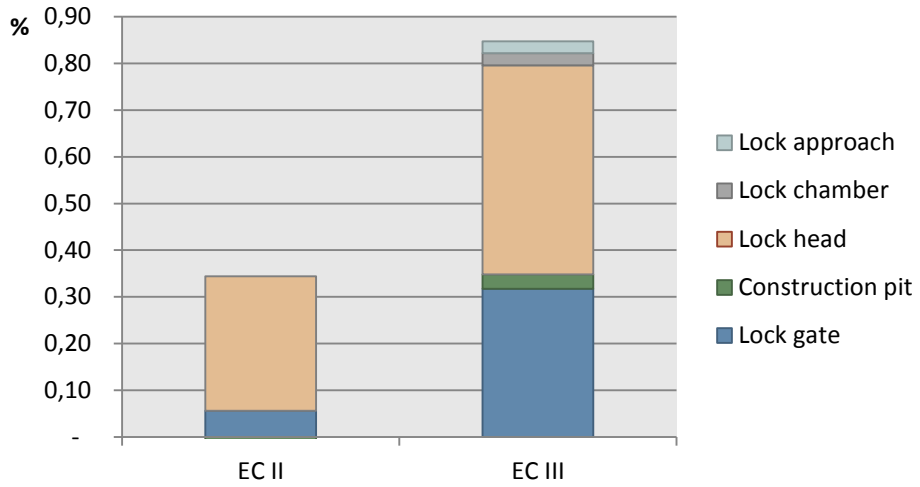


Figure 6-16: Comparison of the standards (on overall costs) with the NEN in combination with CUR III (Cumulative)

The element can also be seen separately. In this case the increase of the individual elements is shown. This principle is illustrated in Figure 6-17. Concerning the reliability class 3 of the Eurocode, it is noticed that the lock gate and lock head both increases significantly (around the 13%). Again, for the comparison in Figure 6-17, the NEN in combination with the CUR III is set as the representative.

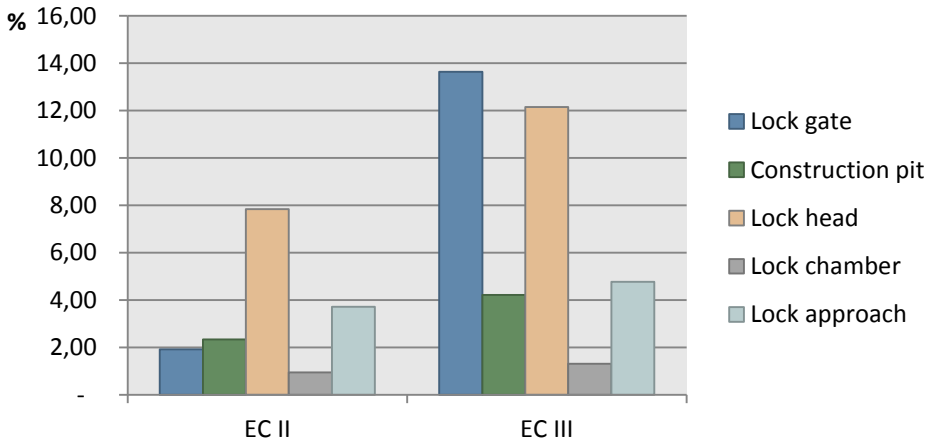


Figure 6-17: Relative increases of the elements compared to the NEN in combination with the CUR III

Remarkable are the high increases of the lock gate and the lock head. During the case, these two elements also draw some attention. The lock gate was strongly influenced by the load partial factor for water level differences. This partial factor involves some limitations with will be discussed in Chapter 7. For the lock head the problem was found in verification method of the Eurocode. The minimum capacity of shear stress for concrete was much lower as the capacity regarding the NEN. Stirrup reinforcement was applied the meet the requirements.

In the beginning of this chapter, four combinations were mentioned of how the standards could be applied. Fort these four combinations, the additional costs (in %) are presented in Table 6-14. An interesting consideration would be between combination 3 and 4. For the last combination only the elements which contribute to the flood defense system are designed on basis of the highest reliability class. By using this combination a profit of 0.29% is taken with respect to the third combination.

Table 6-14: Resulting costs (in %) for the initial combinations

	C1	***C2***	***C3***	***C4***	
	NEN (RC 3)	EC (RC 2)	EC (RC 3)	EC (RC 2)	EC (RC 3)
Lock gate	-	0,04	0,32	-	0,32
Lock head (strength)	-	0,28	0,42	0,28	-
Lock head (stability)	-	0	0	-	0
Lock chamber	-	0	0,02	0	-
Lock approach	-	0	0,02	0	-
Bed protection	-	0	0	-	0
Other	-	0	0,01	0	-
Difference [%]	-	**+ 0.32 %**	**+ 0,89 %**	**+ 0.60 %**	

It is not likely that these results will be same for every other lock. For locks of the same size and dimension, the results will probably be close to each other. But for much larger or smaller complexes the magnitudes will not be equal. However, the deflection will not be large. The decisive elements (from Figure 6-16) are the lock gates and the lock head. These elements will approximately grow along with extent of the lock complex.

In addition, a distinction should be made between the exceedance frequencies of the lock (and the corresponding dike). For high frequencies the partial factor for the water level differences is much lower. This results in smaller differences in costs for the gates. Since the influence of the gates (Figure 6-16) is not negligible, this surely will have some effect on the overall comparison in costs (presented in Table 6-14).

7 Case Study:

Probabilistic determination of the partial load factor and influence coefficient for water level differences

7.1 General

In the LK a determination method for the partial factor for water level differences is prescribed. This determination method consist the load influence coefficient prescribed by the ISO. This influence coefficient is based on probabilistic engineering techniques regarding buildings and bridges, instead of hydraulic structures. Therefore it seems to be necessary to research the applicability of the influence coefficient, which resulted in this case study. This chapter includes a probabilistic research to the influence coefficients and the partial factors for water level differences. In section 5.6 the determination method for this load partial factor is already mentioned:

$$\gamma_w = \frac{u - B \cdot \log(\Phi(\alpha_s \cdot \beta_N) / f_N) - h_b}{u - B \cdot \log(F_{exd}) - h_b} \tag{7.1}$$

In equation 7.1 the influence coefficient is outlined in red. The research involves a probabilistic calculation of a specific structure. During the probabilistic calculation, the influence coefficient can be derived. By adjusting the involved parameters, relations can be observed between the influence coefficient, partial factors and reliabilities. The research involves several steps which are illustrated in Figure 7-1.

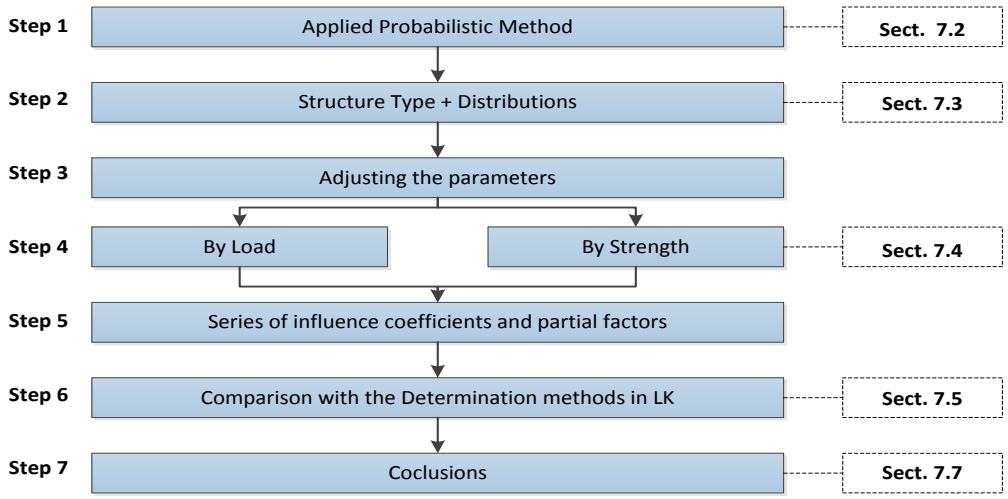


Figure 7-1: Structure of the case study

7.2 Probabilistic Method

In probabilistic design there are three different levels, which were described in Section 2.4.3. The level I approach (semi-probabilistic) is used most often for designing. It makes designing more accessible by the use of partial factors. The level II and level II methods are fully probabilistic. The restriction of the level II method is it considers every variable as a normal distributed variable. The advantage of the Level II method is it directly results in designing points and influence coefficients of the variables. With some changes to the usual Monte Carlo procedure, design points can be found as well. This is especially useful if the discussed reliability function is rather complex. This is not the case for the reliability function treated in this case study. For this reason the probabilistic method which will be applied is a Level II method, which is also called FORM (First Order Reliability Method).

7.2.1 Method Level II (FORM)

The Level II approach makes a distinction in linear and non-linear reliability functions. In this research only non-linear calculations are applied, therefore only this approach will be shortly described. If the reliability function is a non-linear function of a number of independent normally distributed base variables, the distribution of this function is not normal. The function can be approximated by the first two terms of the Taylor-polynomial. The approximation reads:

$$Z = g(\bar{X}) \approx g(\bar{X}_0) + \sum_{i=1}^{n} \frac{\partial g}{\partial X_i}(\bar{X}_0)\,(X_i - X_{0_i}) \tag{7.2}$$

This approximation of Z is linear and according to the central limit theory it is normally distributed. The expected value of the reliability function can be approximated by the expected value of the linearized function:

$$\mu_Z \approx g(\bar{X}_0) + \sum_{i=1}^{n} \frac{\partial g}{\partial X_i}(\bar{X}_0)\,(\mu_{X_i} - X_{0_i}) \tag{7.3}$$

This is also valid for the standard deviation:

$$\sigma_Z \approx \sqrt{\sum_{i=1}^{n} \left(\frac{\partial g}{\partial X_i}(\bar{X}_0)\sigma_{X_i}\right)^2} \tag{7.4}$$

Using the approximations of the expected value and the standard deviation, the reliability index can be approximated.

$$\beta = \frac{\mu_Z}{\sigma_Z} \tag{7.5}$$

Looking for the design point is basically an optimization problem. To find the design point many analytical and numerical approaches can be used. A relatively straightforward method to do this is by first guessing the design point to be the mean value. The obtained β-value is used to determine a new point, in which the reliability function is linearized. In this case the α_i-values are calculated with:

$$\alpha_i = -\frac{\frac{\partial}{\partial X_i} g(\bar{X}^*)\sigma_{X_i}}{\sqrt{\sum_{j=1}^{n} \left(\frac{\partial}{\partial X_i} g(\bar{X}^*)\sigma_{X_j}\right)^2}} \tag{7.6}$$

The new point, in which β- and α_i-values are calculated anew, is determined by:

$$X_i^* = \mu_i - \alpha_i \beta \sigma_{X_j} \tag{7.7}$$

7.2.2 Approximation for non-normally distributed base variables

The water levels are described as a Gumbel distribution. The method II analysis requires variables which are normally distributed. This means that the Gumbel distribution needs to be transformed in a normal distribution. This transformation can complicate an initially simple reliability function. Rackwitz and Fiessler [19] introduced a method, to transform a variable with an arbitrary distribution into a normally distributed variable. This transformation assumes that the values of the real and the approximated probability density function and probability distribution function are equal in the design point. This gives the following system of equations:

$$\begin{aligned} F_X(X^*) &= \Phi\left(\frac{X^* - \mu_X'}{\sigma_X'}\right) \\ f_X(X^*) &= \frac{1}{\sigma_X'}\varphi\left(\frac{X^* - \mu_X'}{\sigma_X'}\right) \end{aligned} \tag{7.8}$$

In which:

φ	= The standard normal probability density function
Φ^{-1}	= The inverse standard normal distribution;
F^{-1}	= The inverse probability distribution of X.
X^*	= Design point
σ_X'	= Standard deviation of the corresponding normal distribution
μ_X'	= Mean value of the corresponding normal distribution

Solving the system of equation gives:

$$\begin{aligned} \sigma_X' &= \frac{\varphi(\Phi^{-1}(F_X(X^*)))}{f_X(X^*)} \\ \mu_X' &= X^* - \Phi^{-1}(F_X(X^*)) \cdot \sigma_X' \end{aligned} \tag{7.9}$$

From the equations (7.8) it appears that the standard deviation and the mean value of the determined normal distribution depend on the design point. Therefore, in the iterative calculation of the design point and of the reliability index, new values for σ_X' and μ_X' must be calculated for every iteration. This is illustrated in Figure 7-2.

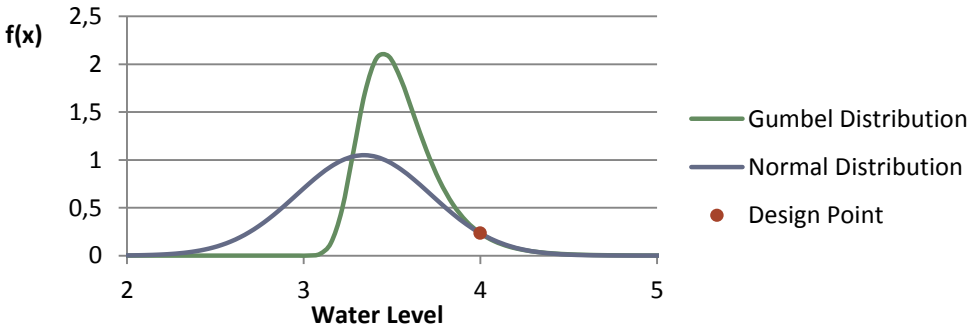


Figure 7-2: Example of transformation from Gumbel to Normal distribution

7.2.3 Overview of the method

The translation should be done during each iteration in the FORM analysis. After each iteration, a new design point is generated. With the new design point, a new normal distribution can be formed. This process is schematized in Figure 7-3.

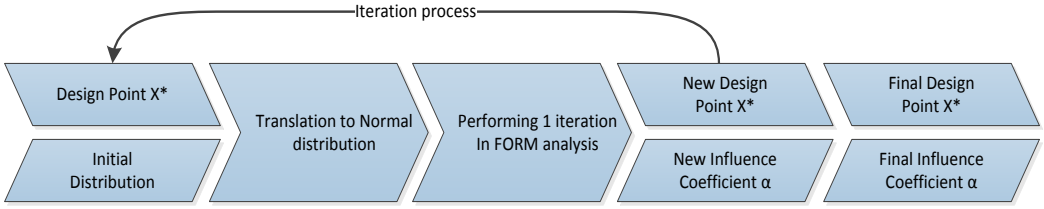


Figure 7-3: Probabilistic method process for an individual parameter in a single FORM analysis

After a number of iterations, the final design point is generated. Each run of the FORM (which are several iterations) analysis results in a final design point for every parameter that is part of the reliability function. The same applies for the influence coefficient. Now, in this analysis, the initial parameters are modified one by one in order to find the relationships between the reliability and the influence coefficients (and design values). This process is schematized in figure 7.4. It starts with a number of initial parameters (X_1 to X_i). For one of these parameters, various values are entered as input of the analyses ($X_{2,1}$ to $X_{2,j}$). These finally results in a list of design values for each parameter and each combination. In this way the design values can be plotted as a function of the reliability index. The same applies for the influence coefficient.

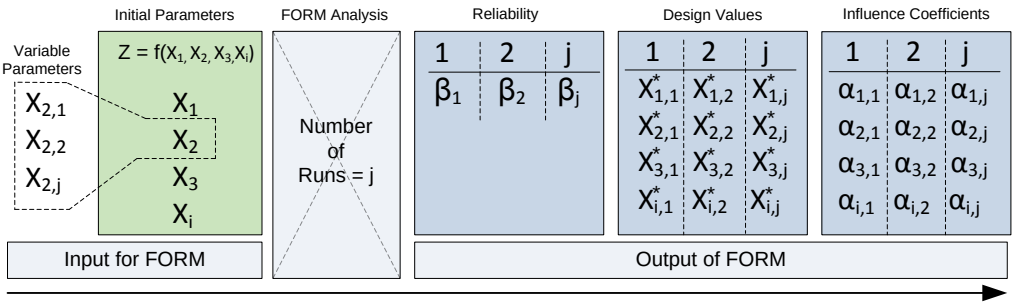


Figure 7-4: Probabilistic process of the FORM analyses for a variable parameter X_2

7.3 Considered Structure and Distributions

The structure that will be analysed in this case is the water retaining gate of a lock. The design of the gate is illustrated in Figure 7-5. It is composed by 7 steel beams which are numbered in the figure. On these beams steel front plates are constructed. On both sides water levels are present, the outer and inner water levels (h1 and h2). This design is taken from the previous case study (chapter 6). The hydraulic boundary conditions are taken from the navigation lock in Nieuwegein (Beatrix sluice).

Table 7-1: Water levels Nieuwegein

Water level h_1 [m NAP]	Frequency of exceedance [1/yr]
6,55	2000
6,35	1000
5,7	100
4,9	10

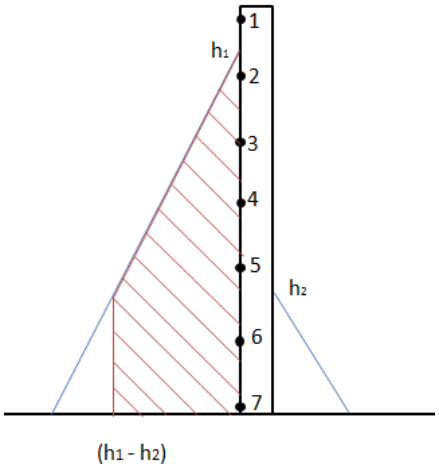


Figure 7-5: Lock gate and corresponding water level data

Failure of the gate will be caused by overloading the critical beam. In this situation the critical beam is appointed by number 6 in Figure 7-5. Due to a too high bending moment the beam will fail. The failure domain of the beam can be described by the following limit state function:

$$Z = f_{y,sp} \cdot W_{el} - \frac{1}{8} \cdot (h_1 - h_2) \cdot \rho \cdot x_{h.o.h.} \cdot l^2 \tag{7.10}$$

In which:

- $f_{y,sp}$ = Yield strength of the steel [MPa]
- $W_{y,el}$ = Elastic section modulus [mm^3]
- h_1 = Water level of the high water side on the critical beam [m]
- h_2 = Water level of the low water side on the critical beam [m]
- $x_{c.t.c.}$ = Length of the plate (equal to the centre to centre distance between the beams) [m]
- ρ = Water density (kN/m^3)
- l = Length of the gates [m]

In the probabilistic method the parameters are defined by distributions. For the water levels a Gumbel distribution is used and for the strength of the steel a Lognormal distribution. The other parameters are set to normal distributions or deterministic values are applied. The Gumbel and Lognormal distribution requires some additional explanations. They will be described in the next two sections. For an overview of the applied distributions see Table 7-4.

7.3.1 Yield strength steel

The yield strength of steel is lognormal distributed. To define the parameter of a lognormal distribution, the mean value of the yield strength of the steel should be defined. The nominal strength value (which are the values published in the most standards) should be converted into a mean value and a standard deviation. This transformation is published by the JCSS (Joint Committee on Structural Safety [18]. The mean value can be found by:

$$f_{y,\mu} = f_{y,sp} \cdot \alpha \cdot e^{u \cdot V} \tag{7.11}$$

In which:

$f_{y,\mu}$	= The mean value of the yield strength of the steel;	[MPa]
$f_{y,sp}$	= The nominal value of the yield strength of the steel;	[MPa]
α	= spatial position factor r (α = 1.05 for webs of hot rolled sections and α=1 otherwise);	[-]
u	= The factor related to the fractile of the distribution used in describing the distance between the code specified or nominal value and the mean value; u is in the range of -1.5 to -2.0	[-]

The lognormal cumulative distribution and lognormal density distribution functions are defined as follow:

$$F_X(X) = \phi_N\left(\frac{Ln(X) - \mu_Y}{\sigma_Y}\right)$$
$$f_X(X) = \frac{1}{X}\frac{1}{\sqrt{2\pi} \cdot \sigma_Y} e^{\left(\frac{Ln(X)-\mu_Y}{\sigma_Y}\right)} \tag{7.12}$$

In which:

$$\sigma_Y = \sqrt{\ln\left(\frac{{\sigma_X}^2}{{\mu_X}^2} + 1\right)}$$
$$\mu_Y = \ln(\mu_X - 0{,}5\sigma_Y^2) \tag{7.13}$$

In which:

X	= Lognormal distributed variable, in this case the strength (can be written as $X = e^Y$)	
Y	= Normal distributed variable Y (Y = ln(X))	
μ_X	= Mean value of the normal distribution	[MPa]
σ_X	= Standard deviation of the normal distribution	[MPa]
μ_Y	= Mean value of the lognormal distribution	[-]
σ_Y	= Standard deviation of the lognormal distribution	[-]

In Table 7-2 an overview is shown for several yield strengths of the lognormal distributions from Eq. 7.11 and 7.12. In this case study the yield strength of 275 is applied as initial nominal yield strength.

Table 7-2: Distribution parameter for several nominal yield strengths

fy,sp [MPa]	fy,μ (μ_X) [MPa]	V	σ_X [MPa]	1- F(fy,sp)	μ_Y [Eq: 7.12]	σ_Y [Eq: 7.12]
235	284	0,07	19,9	0,70%	5,65	0,07
275	**332**	**0,07**	**23,2**	**0,70%**	**5,80**	**0,07**
355	429	0,07	30,0	0,70%	6,06	0,07
460	556	0,07	38,9	0,70%	6,32	0,07

7.3.2 Water level

The water levels are defined as a Gumbel distribution. Instead of the mean value and standard deviation two new parameters will be introduced. The cumulative distribution and density distribution functions are defined as:

$$F(x) = e^{-e^{(-\frac{x-\alpha}{\beta})}}$$
$$f(x) = \frac{1}{\beta} e^{(-\frac{x-\alpha}{\beta})} e^{-e^{(-\frac{x-\alpha}{\beta})}} \quad (7.14)$$

In which:

α = Location parameter [-]
β = Scale parameter [-]

The parameters of the distribution are found by using the water levels data from Rijkswaterstaat [33]. With at least two observation points (of a very low frequency) the Gumbel distribution parameters are defined. This is also explained in Section 3.6.3. In Figure 7-6 the exceedance frequency and the corresponding Gumbel distribution are plotted. By fitting the exceedance frequency with a trend line the parameter for the Gumbel distribution is found.

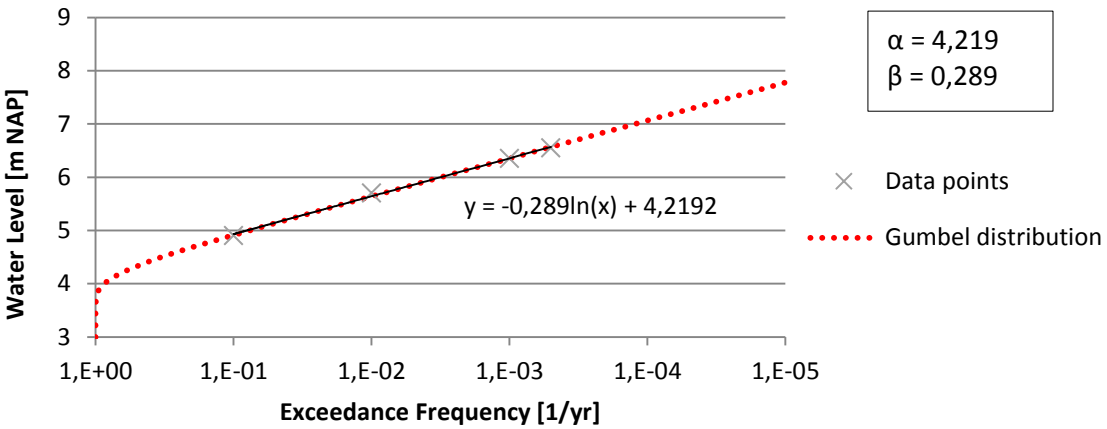


Figure 7-6: Defining the parameter of the Gumbel distribution

In order to find the first translation to the normal distributed function, a design point should be selected. Usually the analyses should start with the mean value of the non-normal distribution. For the Gumbel distribution the mean value is described as:

$$\mu_{Gumbel} = \alpha + 0.5772 \cdot \beta \quad (7.15)$$

By applying the transformation process described in Section 7.2.2, the first transformation can be carried out. The results of the transformation are presented in Table 7-3.

Table 7-3: First transformation from Gumbel to Normal distributions

μ_{Gumbel} (Eq. 7.13)	Gumbel		Normal	
	α	β	μ_i	σ_i
4,52	4,21	0,28	4,46	0,354

7.3.3 Overview of the applied distributions

In Table 7-4 the other variables are defined. Some of them are normally distributed and others are deterministic approached. It must be noticed that the inner water level is described as a normal distributed parameter. These are often channel waters. In theory it wouldn't be correct to consider those as a normal distribution, a Gumbel or Weibull distributions results in a more accurate fit. But because there are no extreme water levels available, it is difficult to compose an accurate distribution. Because the standard deviation is relatively low, the possible error (caused by this assumption) will be low as well.

Table 7-4: Overview of the initial distributions of the FORM analysis

	Unit	Distribution	μ	V	σ
f_y	MPa	Lognormal	332	0,07	23,2
W_{el}	mm^3	Normal	4287000	0,04	171480
h_1	m NAP	Gumbel	4,46		0,35
h_2	m NAP	Normal	-0,40		0,28
ρ	kg/m^3	Normal	10	0,02	0,2
$x_{h.o.h.}$	m	Normal	2	0,02	0,04
l	m	Normal	7,2	0,02	0

7.4 Modifying the parameters

Now the reliability function and the variables are defined, the analysis can start. By adjusting the mean value (μ) of one of the variables, the results FORM calculation will change directly. It would be interesting to see what the effect of this change will be on the load partial factor and the influence coefficient of the water level differences. For this reason the probabilistic calculation is performed for several situations. The following three elements are stated as a variable:

- Beam profile (Strength)
- Nominal yield Strength (Strength)
- Inner Water Level (Load)

Before this parameter will be modified, the FORM calculation will be performed with the initial values presented in section 7.3.3 (Table 7-4). This calculation is performed in Excel and can be found in appendix G. From the calculations follows a series of design values and influence coefficients for each variable. With the characteristic values the partial factor can be defined. See Table 7-5 the results are shown in combination with the distribution parameters presented in previous section.

Table 7-5: Results of the FORM analysis with the initial distributions from Table 7-4

Input		**Output (β = 6.2)**		
	Characteristic value	Influence Coefficients	Design Value (X*)	Partial factor
f_y	275,0 MPa	0,41	275,9 Mpa	1,003
W_{el}	5701000 mm^3	0,25	5335661 mm^3	0,936
h_1	6,55 m NAP (MHW)	-0,84	9,11 m NAP	-
h_2	-0,400 m NAP (HW)	0,15	-0,66 m NAP	-
ρ	10,0 kN/m^3	-0,10	10,00 kN/m^3	1,000
x	2,0 m	-0,10	2,03 m	1,013
L	7,2 m	-0,20	7,20 m	1,000

7.4.1 Modifying the strength parameters by changing the beam

The first modified parameter which will be analyzed is the type of beam (and with that the section modules (W_{pl})). This process is illustrated in Figure 7-4. The input value of the section modules varies for every run of the FORM analysis. In Table 7-6 the relevant results are shown of the FORM analysis by varying the beam profile. They are divided by input values and output values. Only the relevant values are presented.

Table 7-6: Results of the FORM analysis by varying the beam profile

FORM	**Input**			**Output**						
Run nr.	Beam	W_{el} [mm^3]	Δh_{char} [m]	β	$h_{1,d}$ [m NAP]	$h_{2,d}$ [m NAP]	$h_{1,d}$ - $h_{2,d}$ [m]	γ_w	α_{h1}	α_S
1	HE-B400	2884000	6,95	2,94	5,67	-0,64	6,31	0,91	-0,78	0,85
2	HE-B500	4287000	6,95	4,94	7,57	-0,66	8,23	1,18	-0,85	0,87
3	HE-B600	5701000	6,95	6,20	9,11	-0,66	9,77	1,41	-0,84	0,87
4	HE-B700	7340000	6,95	7,26	10,50	-0,66	11,16	1,61	-0,83	0,86
5	HE-B800	8977000	6,95	8,10	11,60	-0,67	12,27	1,77	-0,82	0,84

Some results from Table 7-6 are plotted in Figure 7-7. In these graph the relations of the reliability with the influence coefficients and load partial factor for water level differences are shown. A remarkable observation is the high influence coefficients for the loads (α_S). These coefficients are higher than the ISO recommends.

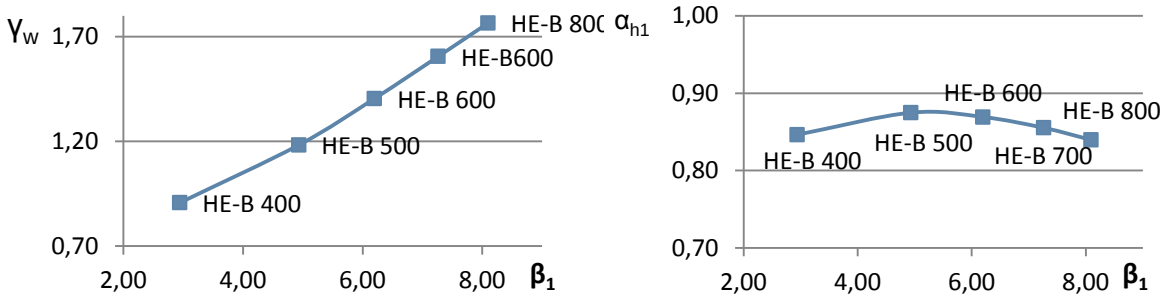


Figure 7-7: Results of the influence coefficients and load partial factor for water level differences. Remarkable are the high influence coefficients. These are much higher as the current applied influence coefficients prescribed by the ISO.

7.4.2 Modifying the strength parameter by changing the yield strength

The second element that is taken as a variable is the yield strength of the steel. In the previous section the distribution of the yield strength of the steel is defined. In Table 7-2 four yield strengths are discussed with their corresponding lognormal distribution. By using each of these strengths in a individual FORM analysis, new results are found, that are presented in Table 7-7.

Table 7-7: Result of the FORM analysis by varying the nominal yield strength of the steel

FORM	Input			Output						
Run nr.	f_y [MPa]	f_μ [MPa]	Δh_{char} [m]	β	$h_{1,d}$ [m NAP]	$h_{2,d}$ [m NAP]	$h_{1,d} - h_{2,d}$ [m]	γ_w	α_{H1}	α_S
1	235	283	6,95	4,19	6,77	-0,65	7,42	1,07	-0,85	0,86
2	275	332	6,95	4,94	7,57	-0,66	8,23	1,18	-0,83	0,86
3	355	428	6,95	6,07	8,94	-0,66	9,60	1,38	-0,85	0,86
4	460	555	6,95	7,17	10,37	-0,66	11,03	1,59	-0,85	0,86

Again, the results of the influence coefficients and load partial factor from the table are plotted in Figure 7-8. Approximately the same influence coefficients are found as in Figure 7-8. Also the relation between the reliability and the partial factor are equal.

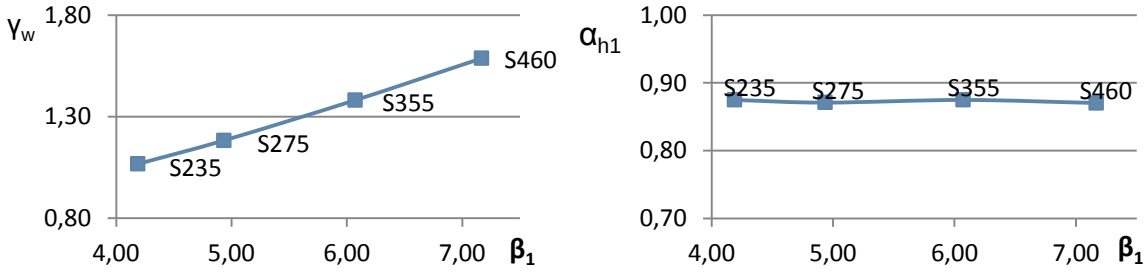


Figure 7-8: Results of the influence coefficients and load partial factor for water level differences. The results are in line with the results from Figure 7.5. Apparently the varying the strength parameter results in no changes of the influence coefficient. Also the increase of the partial factor does not change (almost linear).

7.4.3 Modifying the load parameter by changing the inner water level

In the previous section the strength parameters of the gate were chosen as a variable. In this section the load parameter will be set as a variable. The outer water level will be constant, but the inner water level will vary from +3 NAP to -3 NAP. See Figure 7-9 for the water levels which are considered. In this case different water level differences are created.

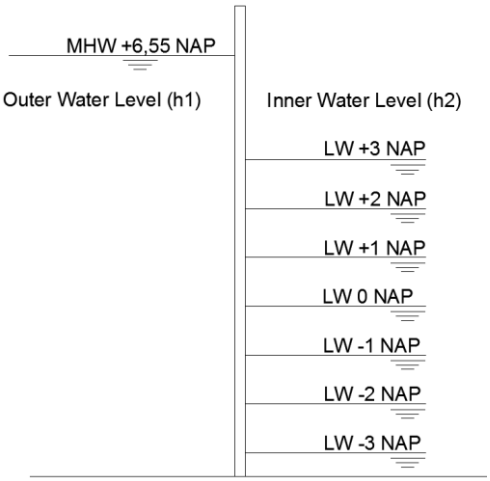


Figure 7-9: Inner water levels which are considered in the analysis

For the strength parameters the initial values are chosen (see table 7.3). The same analysis is carried out as before. It is assumed that the critical beam is loaded with the critical water pressure. The results of the FORM analysis are presented in Table 7-8.

Table 7-8: Results of the FORM analysis by varying the inner water level

FORM	Input		Output						
Run nr.	h_2 [m NAP]	Δh_{char} [m]	β	$h_{1,d}$ [m NAP]	$h_{2,d}$ [m NAP]	$h_{1,d} - h_{2,d}$ [m]	γ_w	α_{H1}	α_S
1	-3	9,55	3,30	5,62	-3,23	8,85	0,93	-0,68	0,75
2	-2	8,55	3,97	6,31	-2,25	8,56	1,00	-0,77	0,81
3	-1	7,55	4,59	7,08	-1,26	8,33	1,10	-0,82	0,86
4	0	6,55	5,16	7,90	-0,26	8,16	1,25	-0,86	0,89
5	1	5,55	5,70	8,77	0,74	8,03	1,45	-0,88	0,91
6	2	4,55	6,21	9,67	1,74	7,93	1,74	-0,90	0,92
7	3	3,55	6,69	10,58	2,74	7,85	2,21	-0,92	0,93

The results in the table are quite revealing. Unlike the other tables, the influence coefficient is strongly depended by the differences in water levels. This is actually quite logical. If the differences in water level are large, the impact of an increase of the outer water level will lead to relatively small changes in difference in water level. Take for example Figure 7-9. If the inner water level is -5 NAP and the outer water level 6,55 NAP, the critical water pressure is 115 kPa. An increase of the outer water level of 1 meter results in an increase in pressure of 10 kPa. This is an increase of less than 10%. In the case the inner water level is +5 NAP, the critical water pressure at the gate is 15 kPa. An increase of the outer water level of 1 meter will lead to an increase of 66% of the critical pressure (to 25 kPa). So the influence of the load is much higher in contrast with high water level differences. The results of Table 7-8 are illustrated Figure 7-10.

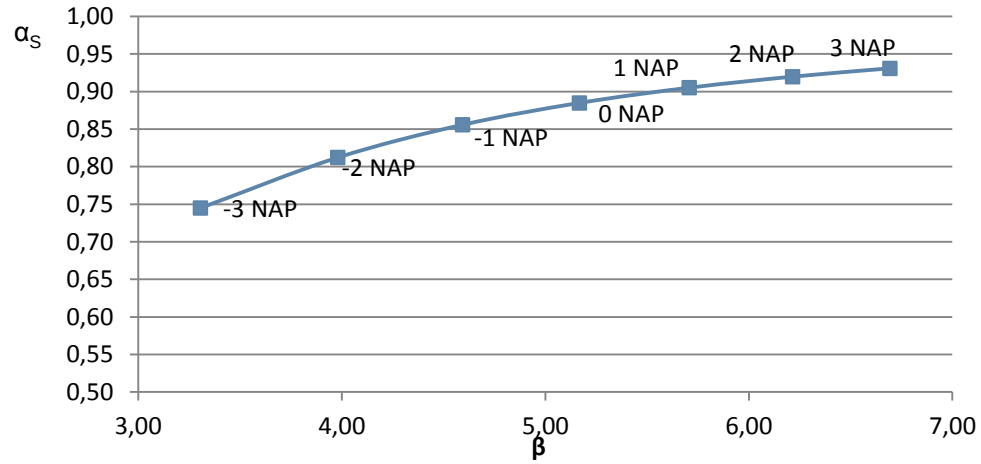


Figure 7-10: Relation water level differences and influence coefficient for loads

So basically, the influence coefficient could be considered as a function of the characteristic water level differences:

$$\alpha(\Delta h) = f(\Delta h_{char}) \tag{7.16}$$

The relation of the influence coefficient and the water level differences is expressed in Figure 7-11. For large water level differences a low coefficients is obtained. But for any situation, the influence coefficient is higher as the currently prescribed influence coefficients by the ISO (α = 0.7). The expressed relation is valid for the current discussed case study. The relation could be different is another distribution of the outer water level is used, or a different type of structure is analyzed.

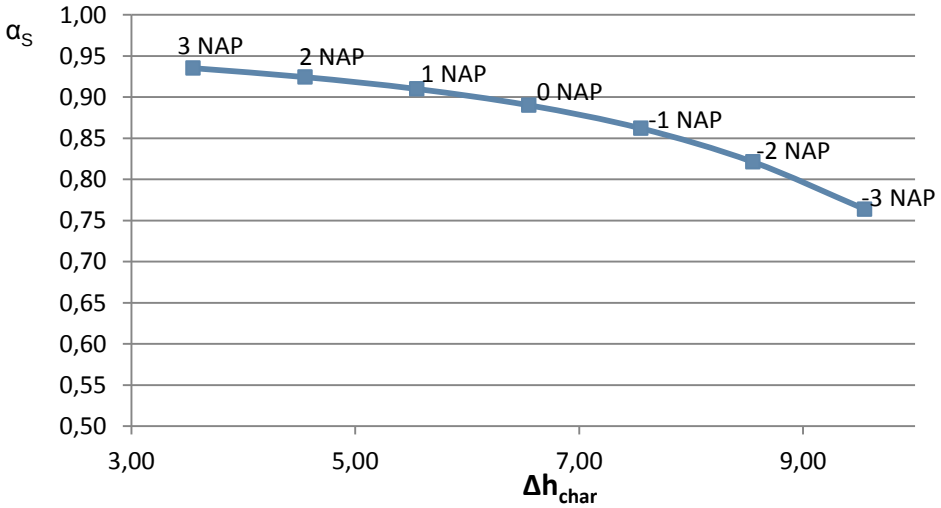


Figure 7-11: Influence coefficient as a function of the water level differences

7.5 Comparison with semi-probabilistic method from LK

To illustrate the effect of the influence coefficient on determining the load partial factor, a comparison is made between the different determining methods in combination with different influence coefficient. The results are applied from the previous section. Those were distinguished by the parameters which were modified. The same distinction is made for this section. The comparisons are made with the detailed method and the simplified method. Both are expressed in Section 4.6. In order to judge the deflections, some boundaries are set for an acceptable deflection. In the Eurocode a requirement is set about the accepted deflection of the influence coefficient.

$$0{,}16 \leq \sigma_S/\sigma_R < 7.16 \tag{7.17}$$

If this requirement is met, the well-known influence coefficients of -0,7 and 0,8 (respectively load and strength) may be used. If not, the influence coefficient of the higher standard deviation (σ_S or σ_R) should be set on $|1{,}0|$, and the influence coefficient for the lower standard deviation on $|0{,}4|$.

The theory says that if the requirement applies, the deviation of the final reliability index (strength and load) is lower than 0.5. In this case study this deviation can be verified, but only for the load side. If the reliability of the load side deflects in one way, it will be compensated with a deflection on the strength side in the other way. Only by combining these two deflection, the total deflection, (strength + load) can be defined and judged (<0,5). For this reason, the final deflection cannot be calculated and a firm conclusion cannot be given. However, the requirement from Eq. 7.15 can be verified, and the deflection on the load side as well. These findings are presented in this section.

7.5.1 Comparison by a modified beam type

In this section the results from Section 7.4.1 (varying the beam type) are compared with the determination method according the LK. In Figure 7-12 the results of the FORM analysis from Table 7-6 are plotted. Those results are compared with the detailed and simplified method of LK. It shows that the simplified method is more conservative as the other results. More interesting would be the accurate fitting of the detailed method with the FORM-analysis, in case the influence coefficient in the detailed method is adjusted to the influence coefficient from the FORM- analyses (from 0,7 to 0,81).

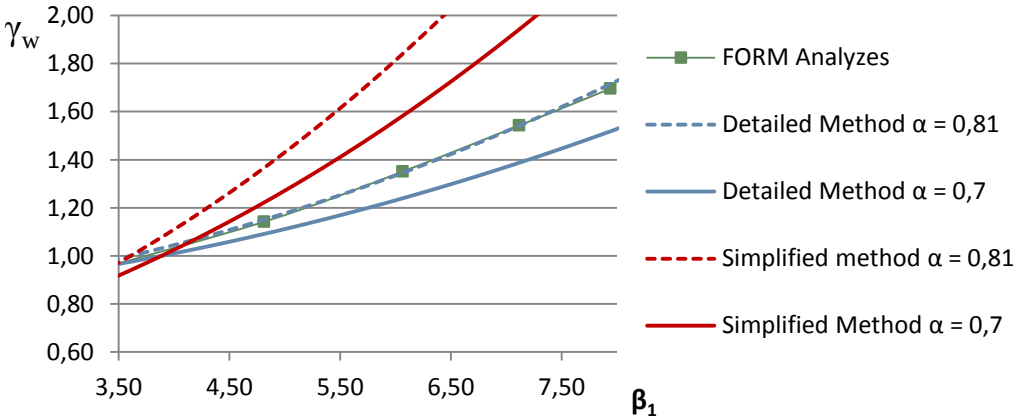


Figure 7-12: Load partial factor calculated by different methods (see Section 4.6) and influence coefficients (in case the type of beam is set as variable)

If now the reliability prescribed by the Eurocode and the LK are read off from Figure 7-12, the related partial factors can be found. This will indicated clear overview of the errors between the different determination methods. This overview is presented in Table 7-9. The resulting partial factors are presented for different used influence coefficient. This is done in order to see the influence of this coefficient. In the first column the influence coefficient is set on -0.7, which is prescribed by the ISO. Remarkable are the results if an influence coefficient is applied of -0,81. In this case the partial factors of the detailed method are identical to the results of the FORM analyzes. It indicates that an influence coefficient of -0.81 ensures a reliable partial factor.

Table 7-9: Vertically read off Figure 7-12. The partial factors are presented that corresponds with the reliability index of the EC and LK

X-axis (β)		Y-axis (γ_w)						
			α_{ISO} = -0,7		Deflection Detailed	α_{h1} = -0,81		Deflection Detailed
	β_1	FORM	Simplified	Detailed		Simplified	Detailed	
RC2	5,10	**1,19**	1,30	1,12	-0,07	1,47	**1,19**	0,00
RC3	4,68	**1,13**	1,19	1,08	-0,05	1,32	**1,13**	0,00
LK	4,42	**1,09**	1,12	1,05	-0,02	1,24	**1,08**	-0,01

Figure 7-12 can also be read off horizontally. For a specific partial factor the corresponding reliability can be found. This is shown in Table 7-10. Each determination method imposes a different reliability index for the same partial factor. The table clearly shows the differences between the imposed reliabilities. Again, the most accurate result is found for the influence coefficient of -0.81 (the reliability corresponds approximately with the results of the FORM analyses).

Table 7-10: Horizontally read off Figure 7-12. The reliability index are presented that corresponds with a specific partial factor

X-axis	Y-axis (β)							
			α_{ISO} = -0,7		Deflection Detailed	α_{h1} = -0,81		Deflection Detailed
γ_w	FORM	σ_S/σ_R	Simplified	Detailed		Simplified	Detailed	
1,05	**4,14**	1,82	4,11	4,41	0,27	3,79	**4,04**	0,10
1,10	**4,49**	1,84	4,32	4,89	0,40	3,97	**4,44**	0,05
1,15	**4,83**	1,85	4,53	5,34	0,51	4,14	**4,81**	0,02
1,20	**5,16**	1,86	4,73	5,75	0,60	4,30	**5,16**	0,00

In Table 7-10 the result of the requirement expressed in Equation 7,17 is presented. It concludes that the requirement is met and the influence coefficients prescribed by the ISO could be used. However, the deflection of the reliability (of the load) is in some case larger than 0,5. This deflection will probably be compensated by an opposite deflection of the strength (were an influence coefficient of 0.8 is applied, instead of the actual influence coefficient). This last step asks for more research, which is missing in this report.

7.5.2 Comparison by a modified yield strength

The same analysis is done with the results from Table 7-7, were the strength of the steel is put as a variable. These are presented in Figure 7-13. Again, this figure shows the results for the detailed and simplified method. They are expressed for two different influence coefficients. One of these coefficients is the currently applied coefficient prescribed by the ISO (-0,7) and the other is the influence coefficient which results in the most accurate fit.

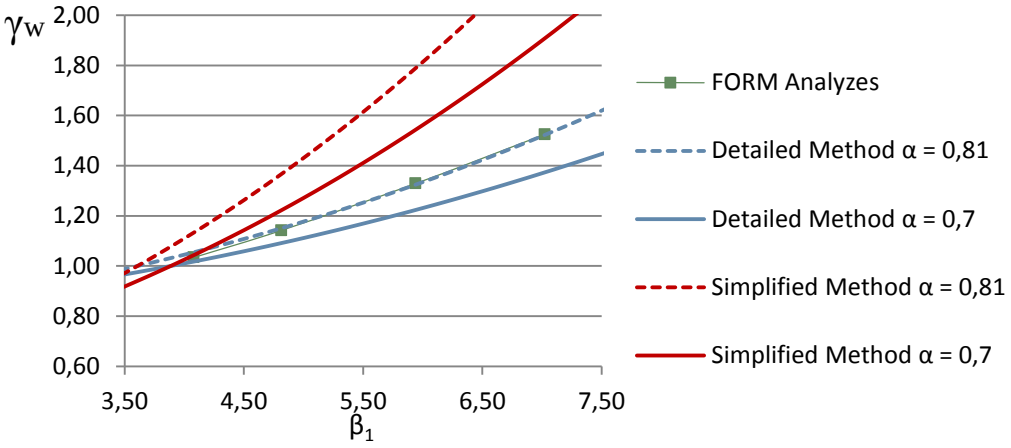


Figure 7-13: Load partial factor calculated by different methods and influence coefficients (in case the steel strength is set as variable)

In this section the same comparison is made is the previous section. It is observed that the results of the FORM analyses from section 7.4.1 and 7.4.2 are almost exactly in line with each other. Due to this, the comparison between the different determination methods will be the same as in the previous section. See Table 7-11 for the results.

Table 7-11: Vertically read off Figure 7-13. The partial factors are presented that corresponds with the reliability index of the EC and LK

X-axis (β)		Y-axis (γ_w)						
			α_{ISO} = -0,7		Deflection Detailed	α_{h1} = -0,81		Deflection Detailed
	β_1	FORM	Simplified	Detailed		Simplified	Detailed	
RC2	5,10	**1,19**	1,30	1,12	-0,07	1,47	**1,19**	0,00
RC3	4,68	**1,12**	1,19	1,08	-0,04	1,32	**1,13**	0,01
LK	4,42	**1,08**	1,12	1,05	-0,03	1,24	**1,08**	-0,01

The same applies if the figure is read off horizontally. The most accurate reliabilities follow from an influence coefficient of -0.81. Again, this is in contradiction of what is prescribed by the ISO/Eurocode.

Table 7-12: Horizontally read off Figure 7-13. The reliability index are presented that corresponds with a specific partial factor

X-axis	Y-axis (β)							
			α_{ISO} = -0,7			α_{h1} = -0,81		
γ_w	FORM	σ_S/σ_R	Simplified	Detailed	Deviation Detailed	Simplified	Detailed	Deviation Detailed
1,05	**4,19**	1,82	4,11	4,41	0,22	3,79	**4,04**	0,15
1,10	**4,53**	1,84	4,32	4,89	0,36	3,97	**4,44**	0,09
1,15	**4,86**	1,85	4,53	5,34	0,48	4,14	**4,81**	0,05
1,20	**5,17**	1,86	4,73	5,75	0,58	4,30	**5,16**	0,02

Again, the requirement from Equation 7.17 is satisfied, which means the influence coefficients prescribed by the ISO seems valid to be used. For a more reliable answer the final reliability deflection should be calculated, which requires a more explicit research to the deflection of the strength.

7.5.3 Comparison by a modified inner water level

In this section the results from Section 7.4.3 (modified inner water level) are compared with the determination method in LK. The results from Table 7-8 are plotted in Figure 7-14. It can be observed that there is a curve in the results from the FORM analysis. This can be explained by the influence coefficients and to the fact that the inner water level is involved in the partial factor. The detailed method will only fit if the right value for the inner water level is used and the corresponding influence coefficient. In Figure 7-14 the lines are added that both represent the detailed method. Of those lines, only the inner water level and the influence coefficient are different. These are mentioned in the legend of the figure. The figure shows that the lines of the detailed method cross the data point from the FORM-analysis for the corresponding inner water level. This implies the accuracy of the detailed method if the correct inner water level and influence coefficient is used.

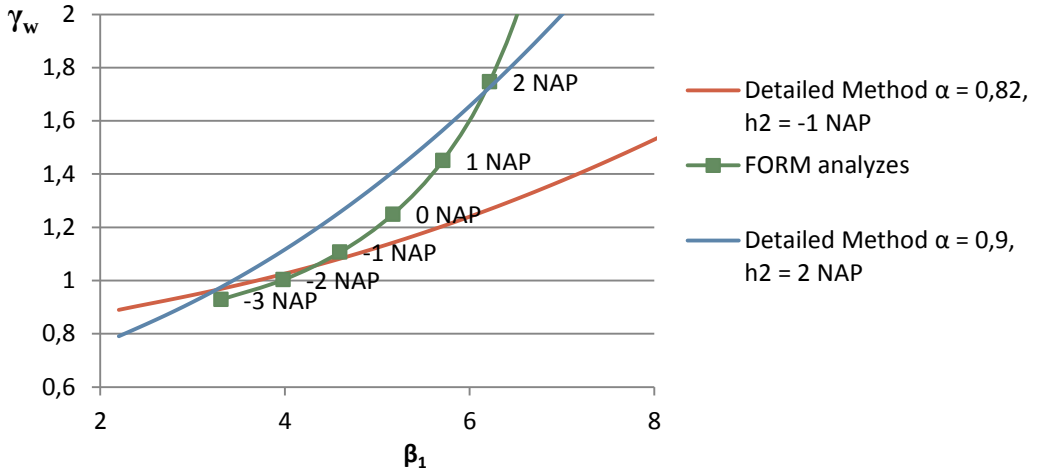


Figure 7-14: Relation inner water levels (and with that the water level differences) and the load partial factor for water level differences.

It is difficult to read Figure 7-14 vertically or horizontally, because the lines of the detailed method do only correspond with a specific inner water level. But the relations between the reliability and the partial factors can be calculated. Those are presented in Table 7-13. It can be noticed that the accurate influence coefficient which should be applied is different for each reliability index. This implies once again that the influence coefficient depends of the characteristic water level difference.

Table 7-13: Vertically read off Figure 7-14. The partial factors are presented that corresponds with the reliability index of the EC and LK

X-axis (β)		Y-axis (γ_w)								
			α = 0,7		α = 0,84		α = 0,82		α = 0,80	
	β_1	FORM	Simpl	Det	Simpl	Det	Simpl	Det	Simpl	Det
RC2	5,10	**1,24**	1,30	1,13	1,53	**1,24**	1,50	1,22	1,47	1,21
RC3	4,68	**1,13**	1,19	1,07	1,37	1,15	1,35	1,14	1,32	**1,13**
LK	4,42	**1,08**	1,12	1,05	1,24	1,09	1,21	**1,08**	1,18	1,07

7.6 The issue for the Simplified Method

In the simplified method the partial factor is standardized to one partial factor, which is explained in more detail in Appendix B. In this method there are three important variables neglected, these are:

- The influence coefficient (α_S)
- The water level differences (Δh_{char})
- Distribution of the water level (Decimeringshoogte, Appendix B)

The first limitation is already mentioned in detail in the previous section. The influence coefficient prescribed by ISO is not based on hydraulic loads. This could result in possible deflections of the final reliability of the structure. It requires more research to the deflection of the strength to find the final deflection of the structure. The last limitation is the standardized scale parameter of the Gumbel distribution. In the case, there is no specific attention paid to this issue. This, because just one type of Gumbel distributed water level is discussed.

The second limitation arises due to the fact that the characteristic water level difference has a significant role in the partial load factor. The general formula of the partial load factor for water level differences is expressed in equation 7.15. The equation implies that the load partial factor increases as a result of smaller characteristic water level differences.

$$\frac{H_{design} - h_{LW,inside}}{H_{MHW} - h_{LW,inside}} = partial\ factor\ for\ water\ level\ differences \qquad (7.15)$$

The practical problem which occurs if the load partial factor for water level differences will be standardized is illustrated in Figure 7.15. In that case a fixed load partial factor is used for any situation (was 1,25 in the period of the LK/NEN). In some situation the design water levels are oversized. For situation A the load partial factor results in the required design water pressures. For situation B, in which the inner water level is slightly lower, the same load partial factor is applied. This results in higher design water pressures as is required. And for situation C the inner water level increases even more, causing a much higher design water pressure as is required.

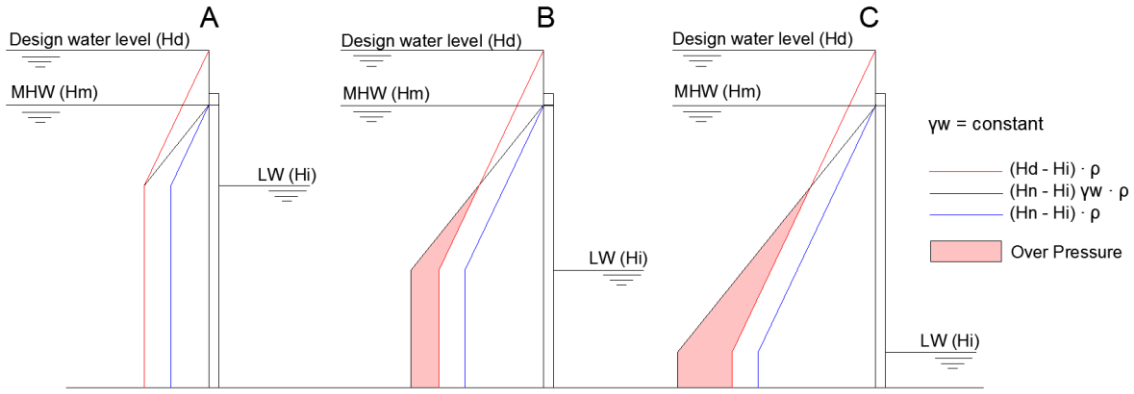


Figure 7-15: [EXAMPLE] Practical problem caused by the simplified method which offers a fixed load partial factor. This example is a bit exaggerated. For situation 1 the fixed load partial factor will lead to an accurate fitting design water pressure. For situation 2 and 3, the same load partial factor will lead to an much large design value as is required. (Hn = MHW; Hi = inner water level; Hd = design water level; y = load partial factor; ρ = water density)

How this limitation is deduced from the analysis in the previous sections is shown in Figure 7-16. In the figure, the results from the analysis in Figure 7-8 are plotted for three different conditions of the inner water level. The lines are derived by varying the yield strength of the steel. It shows the relation between the reliability and the load partial factor. These results are not in line with each other. For the situation in which the water level difference is low (see h_2 = 2 NAP), the required load partial factor for specific reliability is much higher as for a high water level difference (see h_2 = -3 NAP) at the same reliability.

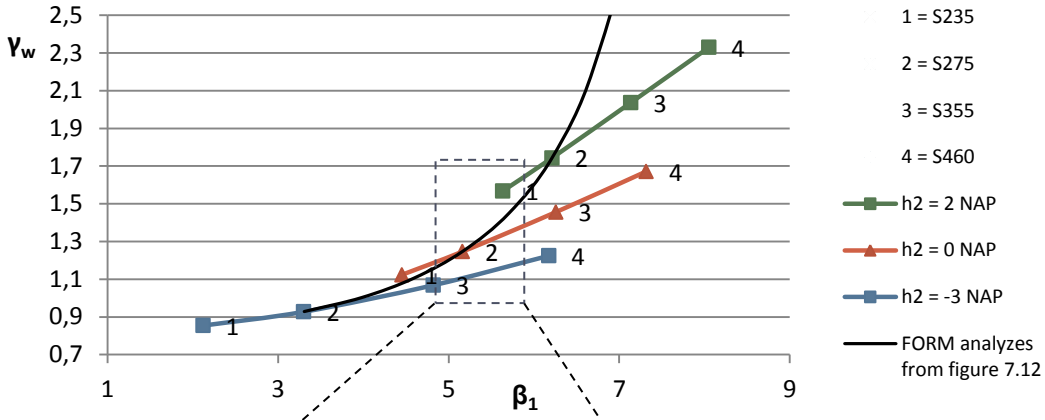


Figure 7-16: Comparison of the results from figure 7.7 for three different inner water level. It clearly shows that the water level difference have a lot of influence on the relation between the load partial factor and the reliability

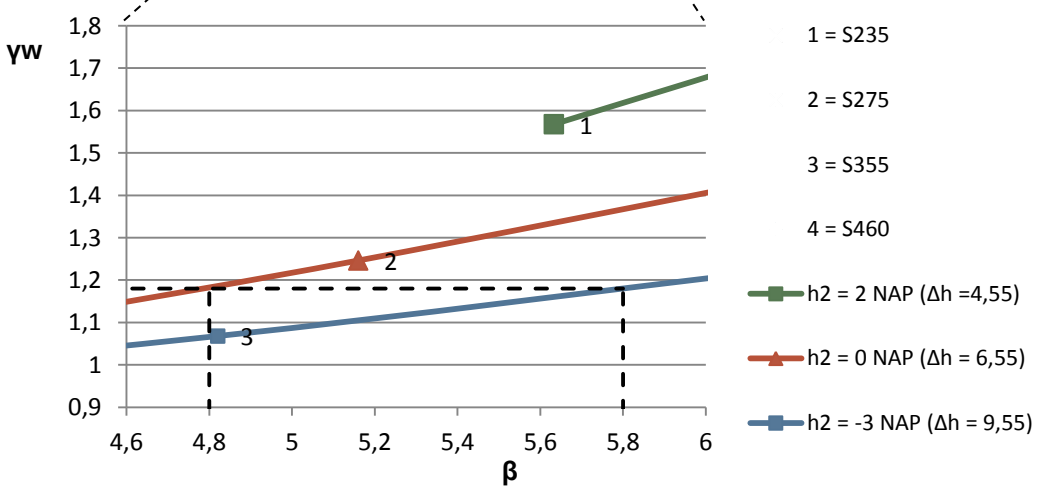


Figure 7-17: This comparison shows the differences in load partial factor between the simplified method and the results from the FORM analyses (for different inner water levels). It shows that the results are far from apart. In the scenarios were the inner water level is lower as 1NAP (Δh = 5,55m), the simplified method is too conservative. There are scenarios in which the simplified method results in a lower partial factor than is required (>1NAP).

It is observed that the lines of the FORM analyses for different water levels do not fit on each other. Different partial factors are derived from the same reliability index. From Figure 7-17 is observed that the errors can be larger than 0,5, which is not acceptable. So, in order to design in an accurate way, it would certainly not be recommended to apply one partial factor for different the water level difference.

7.7 Conclusion

The goal of this case study was the find out if the influence coefficient is applied correctly. From this study the objective can be answered partly. For a more accurate result, more specific research is required to complete the conclusion. The additional research will be explained below. The conclusion is divided into the two determinations method (detailed and standardized).

7.7.1 Detailed method

In Section 7.5 the results of the FORM analyses are compared with the detailed semi-probabilistic method described in Section 4.6. The objective of the analyses was to see if the influence coefficient is applied correctly. In Equation 7.17 a requirement was introduced, by which the correctness of the influence coefficient could be derived. If the requirement was satisfied, the deflection of the reliability index (between the FORM analyses (Method II) and the semi-probabilistic method (Method I)) should be smaller than 0,5. Besides the requirement, the deflection could be calculated as well.

From the comparison in Section 7.5 a larger influence coefficient was found than currently is applied (-0,7) in order to achieve an accurate partial factor. However the requirement from Eq. 7.17 is met in every scenario. This means the deflection of the reliability index should be smaller than 0.5 and the standard influence coefficient (-0,7) can be considered to be correct.

In addition, the actual deflection of the reliability can be calculated as well (to verify if the requirement (Eq. 7.17) is correct). The deflection should be calculated for the partial factors for the loads and for the strength. If a deflection is found for the loads in one way, the deflection of the strength will be the other way around. Only by combining these to deflection the final deflection can be calculated.

In this case study, this deflection is only calculated concerning the load part (partial factor for water level differences). In some situations the deflection was slightly larger than 0,5 (approximately 0.6). It seems realistic to assume that this deflection will be restored to 0.5 (or smaller) by the combining the deflection on the strength side. To be sure, the deflection of the strength should be calculated as well. This requires more research. But basically, it seems that the influence coefficient is used in the correct way for the partial factor for water level differences.

7.7.2 Simplified method

This conclusion can be considered separately from the analyses to the influence coefficient. In the simplified method for determining the partial factor for water level differences, a number of variables are generalized, including the characteristic water level difference. In Section 6.4 is explained that this standardized method results in different reliability indexes for the same partial factor (see Figure 7-17). So, in order to design in an accurate way, it would certainly not be recommended to apply one partial factor for different the water level difference.

8 Case Study:

Economic Benefit of Applying the Eurocode for Locks

8.1 General Approach Case Study

In Chapter 5 two problems were observed concerning the partial factor for water level differences; the influence coefficient and the reliability index. The first is already discussed in the previous Chapter. This case study will research the reliability index (indicated in blue of equation 7.1).

$$\gamma_w = \frac{u - B \cdot \log(\Phi(\alpha_s \cdot \beta_N) / f_N) - h_b}{u - B \cdot \log(F_{exd}) - h_b} \tag{8.1}$$

To compute the optimum reliability (β) of a lock (which is part of the primary flood defense system), the whole flood defense system should be taken into account. This is a quite complex calculation and too comprehensive for this research. For this reason this case study is limited and could be seen as a contribution of answering the question: would the Eurocode be applicable for locks that are part of the flood defense system? In this case the economic influence on the flood defense system is researched.

The case study aims to define the economic benefit of using the Eurocode for lock that are part of the flood defense system. Because of the differences in the fault tree, it would be interesting to analyze the effect of the Eurocode on the flood defense system. It is unlikely to conclude that the economic benefit, of using the Eurocode, would be equal for locks as well as for buildings.

The manner in which the economic benefit is calculated is shown in Figure 8-1.

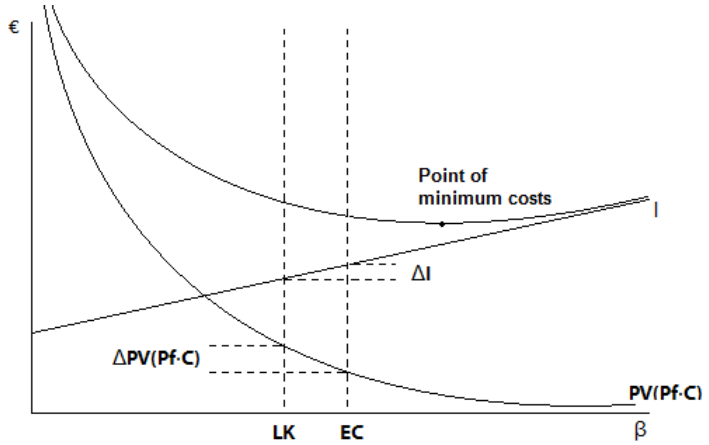


Figure 8-1: Applied approach to research the economic benefit of applying the Eurocode for locks that are part of the Flood Defense System

Basically, the Eurocode lead to an improvement of the flood system. It increases the reliability of the flood defense system. In order to ensure the investment is efficient, the consequences of flooding should be included in the analysis. In Chapter 4 three conceptions of damage were shortly explained: individual risk, social risk and economical risk. In the case where an investment is considered, the economic risk analysis is the most convenient one (see Figure 8-1). In the figure an increase of the investment is indicated (ΔI) and a decrease in risk (ΔPV($P_f \cdot C$)). In case the investment is lower as the decrease in risk, the investment is of economic benefit. In addition, it indicates that the current reliability is on the left side of the optimum point of minimum costs. This situation is also illustrated in Figure 8-1.

The decrease in risk can be expressed as:

$$\Delta PV\left(P_f \cdot C\right) = \left(P_{f,LK} \cdot C\right) - \left(P_{f,EC} \cdot C\right) = \Delta Pf \cdot C \tag{8.2}$$

R

in which:

P_f	= Failure probability of the flood system
$P_{f,LK}$	= Failure probability of the flood system in case the LK is applied
$P_{f,EC}$	= Failure probability of the flood system in case the EC is applied
ΔP_f	= $P_{f,LK}$ - $P_{f,EC}$
C	= Consequence of flooding (Economic Damage) [€]

The change in the total costs of the structure (risk + investments) is defined as:

$$\Delta Total\ Costs\ [TC] = \Delta PV(Pf \cdot C) - \Delta I(Pf) \geq 0 \tag{8.3}$$

In which:

B	= Increase or decrease of the total costs [€]
ΔPV(Pf·C)	= Decrease of the risk [€]
ΔI(Pf)	= Increase of the investment costs ($I_{EC} - I_{LK}$) [€]
I_{EC}	= Total Project Costs of a design based on Eurocode [€]
I_{LK}	= Total Project Costs of a design based on LK [€]

The final goal of the case study is to assess the current dike-rings, to see if the use of the Eurocode minimizes the total costs (risk + investment). If so, it would be recommended to apply the Eurocode. And if not, than it would not be recommended. First the three main elements which are included in the analysis will be briefly discussed.

Consequence

In this analysis the consequence is assumed as an economic damage value (€). It is assumed that failure of one lock results directly in flooding of the area. For the analysis the economic damage values for a selected number of dike-rings are applied. These values are presented in Chapter 3 (Table 3-4). However, in risk analysis this is not realistic, it is a simplified assumption. In risk analyses, the parameter time plays an important role in determining the consequences. It determines to which extent evacuation is possible. The slower the flooding process goes, the more time there is for evacuation. In addition, the flooded area in a dike ring depends on which lock is failing (or which dike area). There are several scenarios how the area could flood.

Investment

The failure probability of the lock in the flood defense system is determined by a select number of elements. Section 3.4.3 describes that not every element in a lock contributes to the probability of flooding (failure of the water retaining function). Besides, the LK as a standard does not affect every element. For a lock there are three

elements that are determinative for the water retaining function, these were the gates; bed protection and the stability of the lock head. The improvement of these elements is seen as the investment that increases the reliability of the lock regarding water retention. In Chapter 6 the additional costs (investment) are calculated for a lock design according the reliability in the Eurocode instead of the reliability in the LK. This particular lock was part of a flood defense system that had an exceedance frequency of 1/1250. The increase in costs (the investment) is shown in Table 8-1. The investments of the other exceedance frequencies are left open. Further on in this case study these values will be applied.

Table 8-1: Investments to go from a design based on LK to a design based on EC. These values are taken from the case study in Chapter 6

	1/1250 [1/yr]	1/2000 [1/yr]	1/4000 [1/yr]	1/10000 [1/yr]
$(I_{EC} - I_{LK})/ I_{LK}$ For $I_{LK} = 100 \cdot 10^6$ €	+0.32%	-	-	-
$I_{EC} - I_{LK}$ (€)	320.000	-	-	-

Fault tree approach

In this analysis the influence on the failure probability of the dike-ring is exclusively discussed. A dike-ring involves of several underlying failure mechanisms. One of these mechanisms is failure of the lock, and one of the underlying failure mechanisms of the lock is failure due to strength or stability. This is the level were the Eurocode get involved. The use of the Eurocode results in a decrease of the failure probability of the dike-ring. The process how the increase in failure probability is obtained is presented in Figure 8.2. The process is divided in 3 steps. First the influence on a single lock is analyzed. Secondly a number of N locks are researched. And finally the dike sections are involved in the analyses. Basically a bottom-up approach is maintained. In addition, the sensitivity of all the involved elements will be analyzed. Finally, it is tried to generate a reasonable decrease of the failure probability, which subsequently is used to determine the decrease in risk.

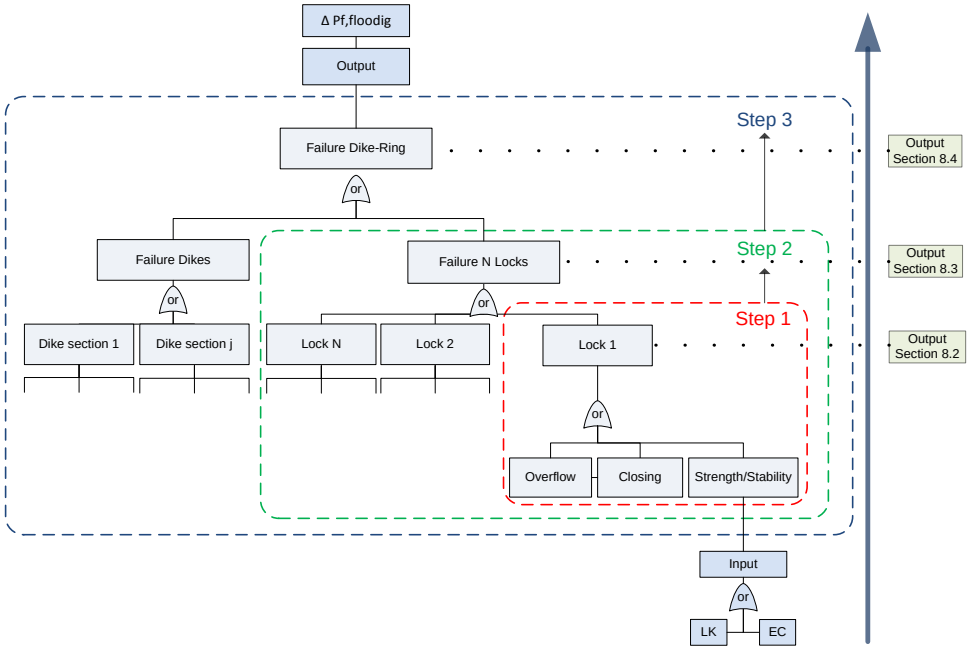


Figure 8-2: Structure of determining the decrease in failure probability

8.2 Influence of EC on one Lock

The first step which will be discussed is the situation in which a single lock will be the top event of the fault tree (see Figure 8-3). From flooding point of view, a fault tree can be composed with all three failure mechanism.

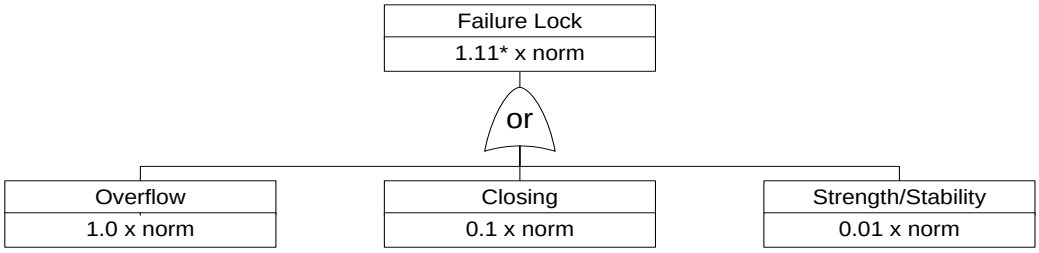


Figure 8-3: Scenario: single lock (* if correlation is not taken into account)

In Figure 8-3 the top event is noted as the sum of the underlying failure mechanisms. This will be the case if all mechanisms are independent. But that is not true. There is a certain correlation between these failure mechanisms. In Appendix I the full calculation is expressed and the correlations between the mechanisms are included. In Chapter 4.3 the theory behind the correlation factor is applied. Also the correlation values are indicated. These are:

ρ(Overflow, Strength/Stability) = 0,63
ρ(Overflow, Closing) = 0
ρ(Strength/Stability, Closing) = 0

With the information from the fault tree in Figure 8-3 and the correlations between the mechanisms, a relation can be sketched between the reliability of the mechanism 'strength and stability' and the failure probability of the lock. This relation is illustrated in Figure 8-4. It can be noticed that there is a large difference between the failure probability of the lock from flooding point of view and the failure probability of the mechanisms 'strength and stability'. When the reliability index (of mechanism S/S) passes a certain value, the reliability of the lock as a whole becomes constant. From that point the influence of increasing the reliability index (of S/S) would be almost zero. For lower reliability indexes (<4), the overall failure probability of the lock will increase immediately.

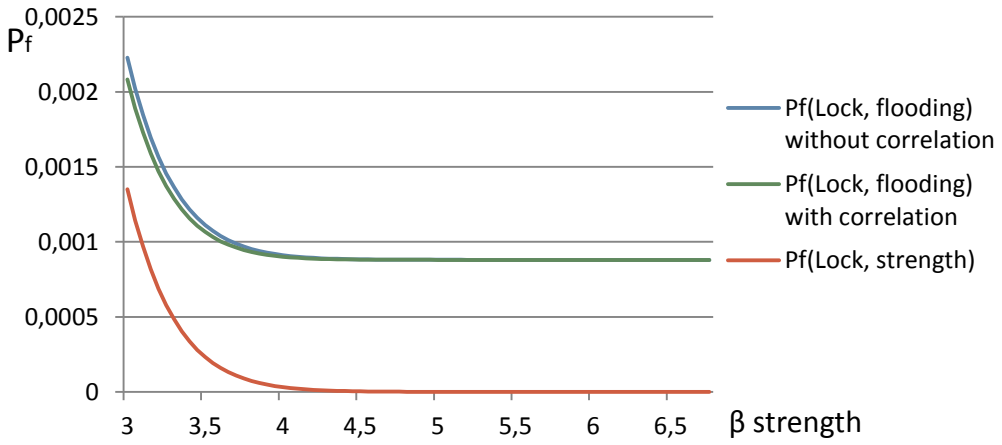


Figure 8-4: Relation reliability S/S and failure probability of the lock (F_{exd} = 1/1250)

This limit of influence on the failure probability of the lock is even more clearly visible in Figure 8-5. In this figure the failure probabilities of the lock and strength are converted to the reliability index. It clearly shows that the reliability is limited to a fixed value. This is caused due to the presence of other failure mechanisms which has a much higher failure probability as the failure probability due to strength and stability. These other mechanisms are failure of closing and overflow. Because of this, the failure probability of the top event (flooding) will be of the same order of magnitude as the failure probability of overflow, and is not being influenced by a change in strength and stability.

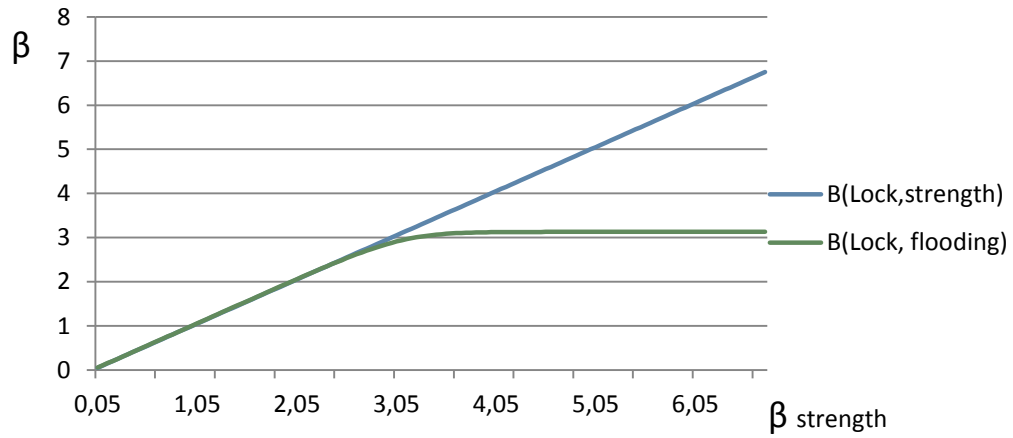


Figure 8-5: Relation reliability S/S and the reliability of the lock (F_{exd} = 1/1250)

Now the Eurocode introduced a new reliability class (RC3) which in principle should also be used for strength and stability design calculations of hydraulic structures. This standard replaces the standards in LK (more critical as the NEN-standard). The Eurocode reliability class 3 maintains a reliability index of 4.3 and is based on a reference period of 50 years. The standards in LK are based on a reference period of 1 year. In order to compare these standards, they should be converted to the same reference period. In Table 8-2 an overview of the indexes and related standard in presented.

Table 8-2: Probabilities of failure and reliability index

	Eurocode	**LK**			
	RC 3	1/1250	1/2000	1/4000	1/10000
$\beta_{f,1yr}$ [-]	**5,2**	**4,31**	**4,42**	**4,56**	**4,75**
$\beta_{f,50yr}$ [-]	4,3	3,35	3,48	3,66	3,89
$P_{f,1yr}$ [1/yr]	**9,96E-08**	**8,00E-06**	**5,00E-06**	**2,50E-06**	**1,00E-06**
$P_{f,50yr}$ [1/yr]	8,54E-06	4,00E-04	2,50E-04	1,25E-04	5,00E-05

In order to see what the effect of the Eurocode will be, a fault tree calculation will be made for two situations. In the first situation the index according LK is used and in the second the situation the index according the Eurocode is used. The results of this calculation can be compared.

In the situation the lock in Figure 8-3 is discussed with an exceedance frequency of 1/1250, the failure probability will be:

Failure Top Event (Lock in LK): $8.845 \cdot 10^{-4}$ ➔ **1/1130 [1/year]**

The importance coefficient of the failure mechanisms are expressed in Table 8-3. Overtopping is the dominant failure mechanism whit an importance range of 90.09 percent. This means that of the 1000 failures (Monte Carlo Analysis) 90.09 percent of these failures is caused by overtopping. The failure mechanism 'Strength and Stability' has instead a much lower factor of importance.

Table 8-3: Factors of importance according LK (F_{exd} = 1/1250)

Event	Failure contribution [1/yr]	Importance
Overtopping	8. 00E-004	90,09 %
Closing	8. 00E-005	9,01 %
Strength and Stability	8. 00E-006	0,90 %

If the same calculation would be made using the reliability index according the Eurocode (from Table 8-2), the following failure probability is found:

Failure Top Event (Lock in EC): $8.799 \cdot 10^{-4}$ ➔ **1/1136 [1/year]**

As a result of increasing the reliability of the mechanism 'Strength and Stability' (and lower its failure probability), the factor of importance will decrease. In Table 8-4 the importance value are presented.

Table 8-4: Factors of importance according EC

Event	Failure contribution [1/yr]	Importance
Overtopping	8. 00E-004	90,91 %
Closing	8. 00E-005	9,09 %
Strength and Stability	9. 96E-008	0,01 %

Between the two standards, there is a small difference in the resulting failure probability of the top event (lock). In Table 8-5 an overview is expressed of both failure probabilities. The difference is expressed with respect to the initial failure probability (from LK). It results in a decrease of 0.52% (=$\Delta P_{f,\%}$), which is expressed as:

$$\Delta P_f = P_{f,LK} - P_{f,EC} \tag{8.4}$$

$$\Delta P f_{\%} = \frac{P_{f,LK} - P_{f,EC}}{P_{f,LK}} \cdot 100\% = \frac{\Delta P_f}{P_{f,LK}} \cdot 100\% \tag{8.5}$$

In which:

$P_{f,LK}$ = Failure probability in case the LK is applied [1/year]
$P_{f,EC}$ = Failure probability in case the Eurocode is applied [1/year]

The conclusion for the situation as is indicated in Figure 8-3 is; the influence of the Eurocode will be and decrease of 0.5% of the failure probability, which is approximately 4.67E-06.

Table 8-5: Comparison of the resulting failure probabilities for both standards

	LK	Eurocode	Δ Pf (eq.8.4)	ΔPf% (eq. 8.5)
Pf(lock, flooding) [1/yr]	$\frac{1}{1130}$	$\frac{1}{1136}$	4.67E-06	**0.52 %**

8.3 Influence of EC on Several Locks

In this section the number of lock will be extended from one to a variable number of locks. In the fault tree the locks are linked with each other with an 'or' port. This means the top event of the fault tree will be failure of one of the lock. The fault tree is illustrated in Figure 8-6.

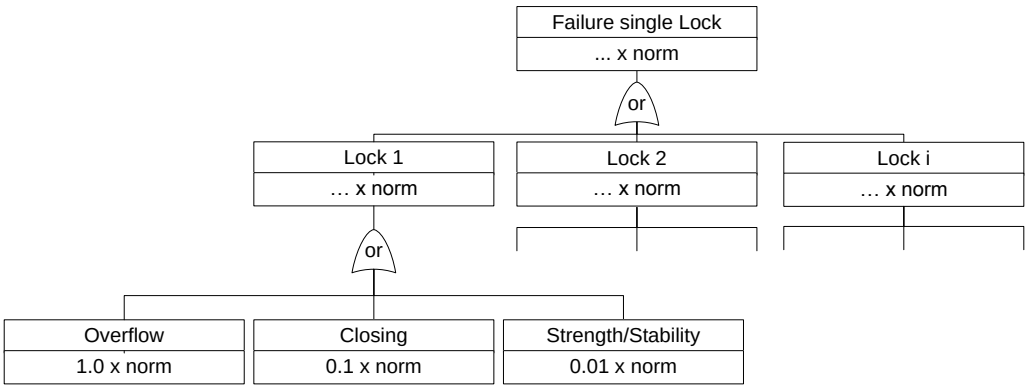


Figure 8-6: Fault tree for scenario: several locks

The failure mechanisms are the same as already discussed before. In this situation the relation between the two single locks should be defined. The correlation between two lock is already discussed in Chapter 4. The correlation depends on the correlated parameters which play a role in the failure mechanism of the lock. From Chapter 4 (and Appendix I.2) are found:

ρ (Overflow$_i$; Overflow$_j$) = 1
ρ (Strength/Stability$_i$; Strength/Stability$_j$) = 0,49
ρ(Closing$_i$; Closing$_j$) = 0
ρ (Overflow$_i$; Strength/Stability$_i$) = 0,63

The calculation structure of the failure probability of a number of locks is shown in Figure 8-7. First the failure probability for a single mechanism is calculated. Thereafter the different mechanisms are combined to one top event failure probability.

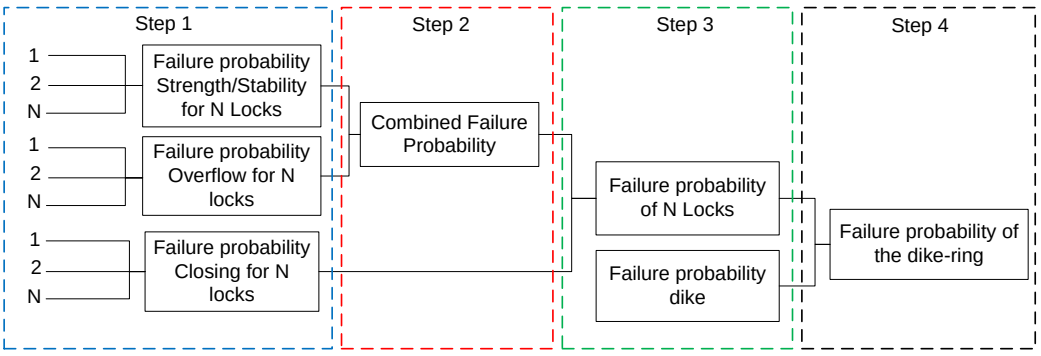


Figure 8-7: Calculation structure of the failure probability of N locks

The calculation and comparison that is made in Section 8.2 can now be performed for a number of locks. The influence of the Eurocode takes place in the mechanism strength and stability. If more locks are applied, the influence of this mechanism could be higher. This depends on the correlations that are applied. If the correlation of the

strength between two different locks is low, and the correlation of the overflow between two different locks is high, then the influence of the strength will be higher as the number of lock rises. In Table 8-6 the importance coefficients are presented in case of a different number of locks. It is clearly shown that the important coefficients for overflow decreases with a higher number of locks. The coefficients for strength and closing increases if the number of locks increases. This can be explained by the correlation parameters that are applied. Regarding the mechanism overflow, it is assumed that locks are fully interdependent. Concerning the mechanisms closing and strength, the locks are not completely interdependent.

Table 8-6: Effect of the number of lock on the importance coefficients

	Importance coefficient					
	LK			EC		
N	Overflow [%]	Closing [%]	Strength [%]	Overflow [%]	Closing [%]	Strength [%]
1	90,09	9,01	0,90	90,90	9,09	0,01
2	82,78	15,67	1,55	84,06	15,92	0,02
3	77,15	20,80	2,05	78,74	21,23	0,03
4	72,68	24,87	2,45	74,48	25,49	0,03
5	69,05	28,18	2,77	70,99	28,97	0,04
6	66,04	30,92	3,04	68,08	31,88	0,04

A high importance coefficient indicates a large influence on the structure. Hence, if an element with a high importance coefficient undergoes a change in reliability, this will have a large effect on the overall reliability of the structure. Conversely, if an element with a small influence coefficient undergoes a change in reliability, the reliability of the structure will be only slightly affected. Such a change happens if the Eurocode is applied instead of the LK. In Figure 8-8 the influence on the failure probability of the lock(s) (of the fault tree in Figure 8-6 and expressed in equation 8.3) is expressed in relation to the number of locks and the exceedance frequency of the locks. If one lock is used, the result corresponds with the calculation made in table 8.5 (4.7E-06). If more locks are involved in the system, the decrease in probability will increase (around 3.1E-05 for 6 locks). Therefore, the number of locks will certainly play a role for magnitude of the influence.

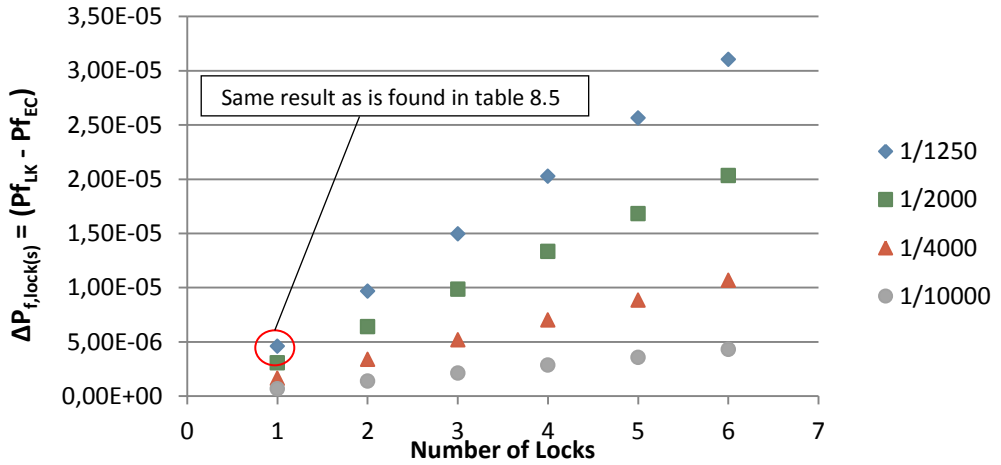


Figure 8-8: Influence on overall failure probability (ΔPf, defined in eq. 8.3) in relation with the number of locks

8.4 Influence on the Dike-ring

In this section the system will be extended by including the failure probability of the dike. Usually the dikes are divided in sever sections which each had its own failure mechanisms. Due to the length effect, the sections are (probabilistically seen) connected with each other. To elaborate this in detail (like is done for the locks), al lot of work is required. Because the focus in this report is set on the hydraulic structures (the locks), the dikes are considered as one mechanism and is included in the research as a variable. In Figure 8-9, the fault tree for this scenario is shown.

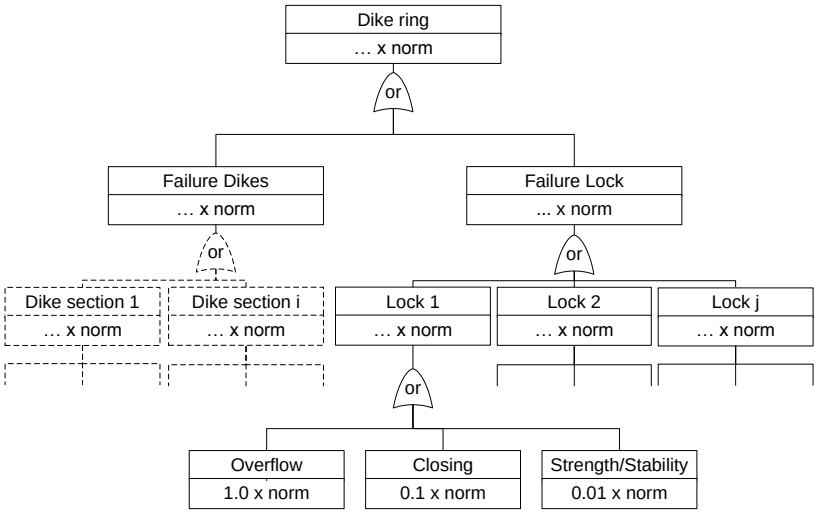


Figure 8-9: Dike ring for example probabilistic approach

In Table 8-7 the failure probabilities for each mechanism are linked with the sections of the dike system. For the dike sections, an overall failure probability is assumed. For the locks, the well-known failure probabilities according LK are applied. An overview is expressed in Table 8-7.

Table 8-7: Failure probabilities expressed by section and failure mechanism

Section	Overtopping	Technical	Closing
Dike 1 Dike 2 Dike i	Pf,dike		
Sluice 1	$1 \cdot F_{exd}$	$0.01 \cdot F_{exd}$	$0.1 \cdot F_{exd}$
Sluice j	$1 \cdot F_{exd}$	$0.01 \cdot F_{exd}$	$0.1 \cdot F_{exd}$

By including the dike another variable is added to the analyses. In the previous section the decrease of the failure probability was calculated by just consider the lock. Now the dike is included, the decrease of the failure probability of the dike ring can be calculated. This decrease will be calculated in this section and is defined as:

$$\Delta P_{f,dike-ring} = P_{fdike-ring,LK} - P_{f,dike-ring,EC} \quad (8.2)$$

The failure probabilities are calculated based on the fault tree in Figure 8-9. The influence of the failure probability of the dike-ring is expressed in Figure 8-10 and Figure 8-11.

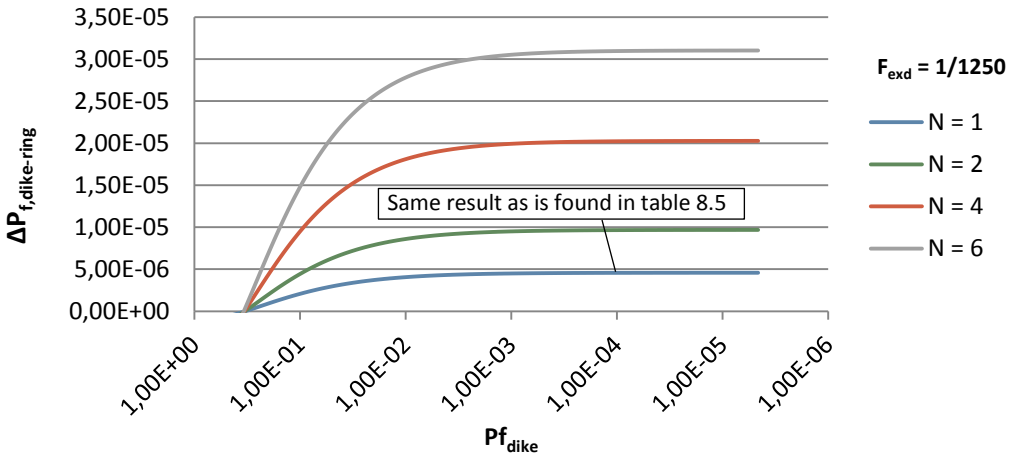


Figure 8-10: Relation failure probability of the dikes and the number of locks (for exceedance frequency = 1/1250)

In Figure 8-10 the exceedance frequency of the locks is fixed at 1/1250. The number of locks and the failure probability of the dikes are presented as a variable. From the figure it can be seen that a higher number of locks increase the influence of the Eurocode on the failure probability of the flood system. The second variable, the failure probability of the dike, plays also an important role. In the case of a relatively high failure probability of the dikes (higher than for the locks), the safety of the flood system completely depends on the dikes. Therefore, the effect of using the Eurocode will be almost zero. If the dikes have a much lower failure probability than the lock, the safety of the flood system depends fully on the locks. In this case the results from the previous section (Table 8-5) are valid.

In Figure 8-11 the same results as in Figure 8-10 are presented, but the fixed variable is changed. It is observed that the maximum decrease of the failure probability of the dike-ring is reached at a relative low failure probability of the dikes. From a failure probability of 0.001, the maximum is almost reached. From there on, the locks are more critical than the dike. Usually, dikes have a lower failure probability than 0.001. Therefore, it can be assumed that the failure probability of the dikes is an irrelevant variable in this research. From now on this variable is set on 0.001 or lower.

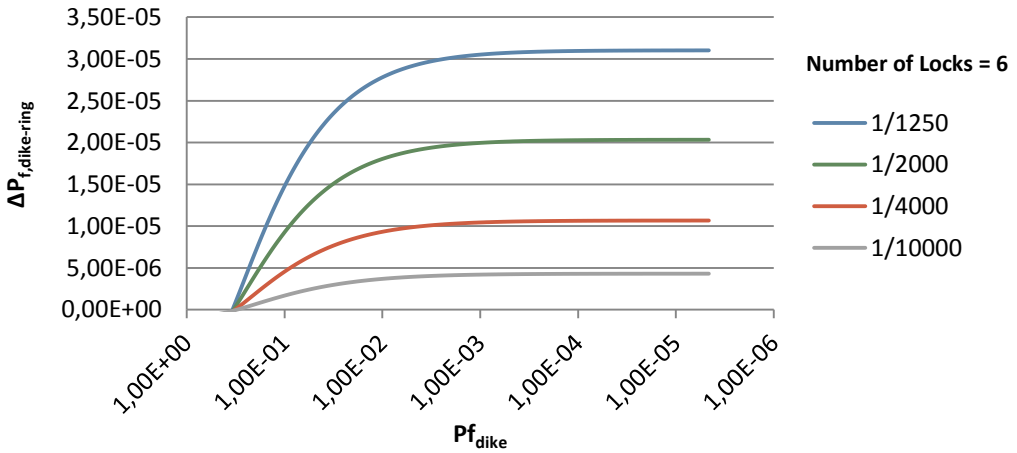


Figure 8-11: Relation failure probability of the dikes and the exceedance frequency of the locks (with N = 6)

8.5 Influence on the Risk

In this section the consequence of flooding is added to the analysis in order to find the influence on the risk. The consequence is considered as a fixed economic damage value of the corresponding dike-ring. These values can rise up to billions of Euros. By including the consequence, a fourth variable is added:

- Number of locks (N)
- Exceedance frequency of the locks (exp: 1/1250)
- Failure probability of the dikes (Pf,dike)
- Consequence of flooding (C)

From the previous section was explained that the failure probability of the dike is set on 0.001 or lower. In Figure 8-12 is the decrease in risk calculated for the situation where the economic damage is set on 100 billion Euros. This is a realistic value but is just an indication to observe the relations between the variables.

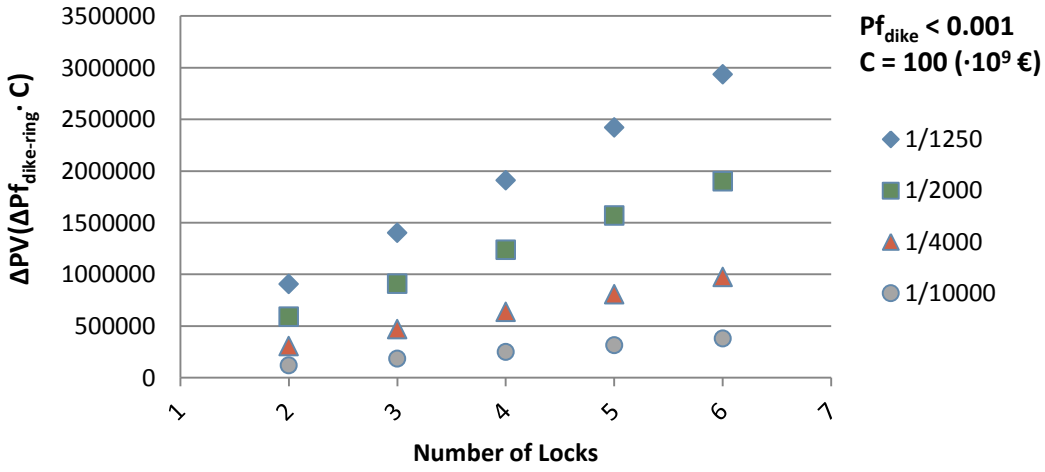


Figure 8-12: Effect of the EC on the risk of flooding

The observations are as they were expected. The more locks are present, the larger the decrease in risk. For exceedance frequency of 1/1250 and a number of lock of 6, the decrease is approximately 3 million Euros. If the required investment is lower than 3 million Euros, the investment is not of economic benefit. This consideration will be analyzed in the next section.

8.6 Economic Benefit of using the Eurocode

Now the risk is defined and calculated, the investment should be involved in the analyses. From there on the economic benefit can be computed. In Figure 8-13 the total costs (Equation 8.3) were expressed as a function of the possible required investments. This graphic is just an indication to illustrate in which scenarios the application of the Eurocode will be effective and which not. Obviously if the investment is relatively low, the result is favorable. High investments do not compensate the decrease of the risk. This turning point occurs at an investment of about 0.5 million. This point is an interesting given. It decides whether the investment is profitable of not. A remarkable observation is that the turning point is relatively independent of the number of locks. The lines in the figure cross each other in the turning point. So basically, from this observation it can be concluded, that the number of locks does not affect the decision of whether the investment is profitable or not. However, it does influence the range of the profit.

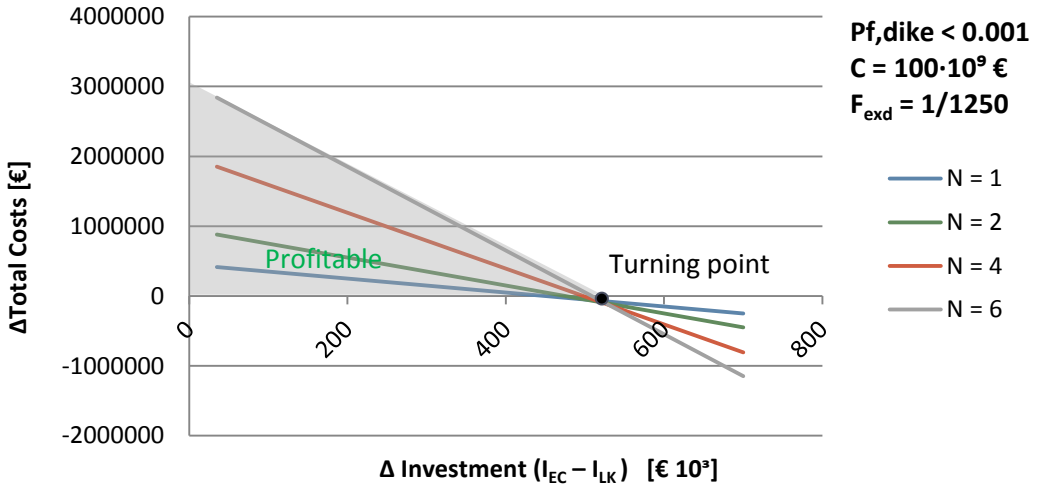


Figure 8-13: Effect on the total costs (investment + risk as a function of the investment

The same figure can also be presented in case the consequence (or economic damage) of failure is chosen as a variable (see Figure 8-14). Again, the turning points for the different number of locks are about equal.

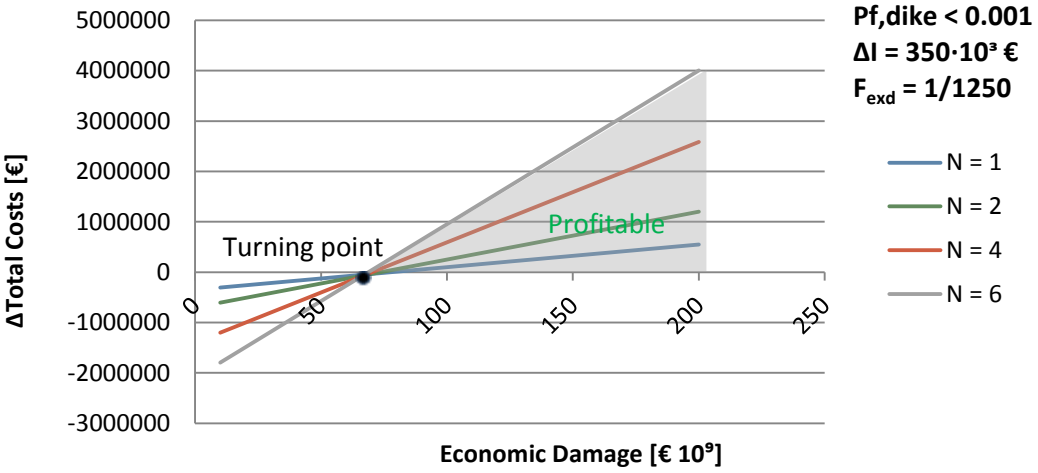


Figure 8-14: Effect on the total costs (investment + risk) as a function of the consequence

From these graphics, it became clear that the number of lock is not a critical factor in the system. It does not influence the turning point significantly. Instead, only the consequence and investment do influence to turning point. In each of the upper two graphics one of the two variables is set as a fixed value. Interesting is now to combine the two variables into one graphic. This relation is shown in Figure 8-15. If the investment is too high or the consequences too low, the investment is non-profitable. At the other hand, low investments and high consequences lead to profitable situations. The figure requires a lot of information of the system to use it properly. As already said, the consequences in this case had been simplified to one economic value, in which it normally should be seen as a process in time. In addition, the investment is a value which will be different for each structure and is also difficult to define.

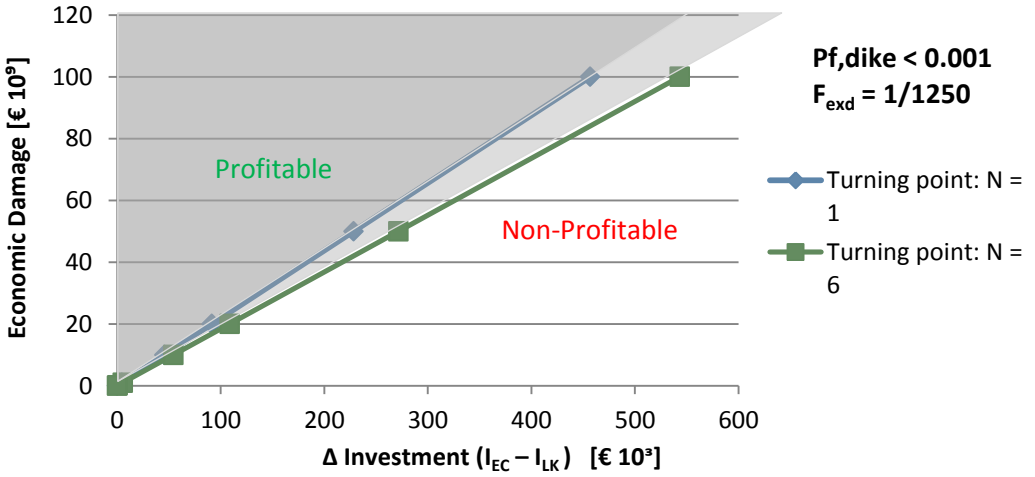


Figure 8-15: Profit boundary as a relation of the investment and consequence. This figure shows for which situation the use of the Eurocode on locks (in the flood defense system) is a success, given the information about the consequence of flooding of the protected area and the corresponding investment. These two facts are difficult to estimate for a lock.

8.7 Assessment of the Current Dike-Rings

In this section some of the current dike-rings will be assessed by using graphic from Figure 8-15. It would be interesting to know on which side from Figure 8-15 the dike-rings in the Netherlands are located, in the profitable or non-profitable area. In section 3.4 (Table 3-4) a number of dike rings are mentioned. In the same table the corresponding economic damage is presented. If these values (for the dike rings with an exceedance frequency of 1/1250) are inserted in Figure 8-15, it could be observed if the applicability of the Eurocode for the dike rings is, form the economic point of view, the right thing to do.

It only requires also a good estimation for the required investment. The case study from chapter 6 is based on a lock which is part of a dike ring with an exceedance frequency of 1/1250. From Table 8-1 follow that concerning this specific lock the investment is about 300.000€. In Figure 8-16 the data of the dike rings are processed. It obviously shows that the dike rings with exceedance frequency of 1/1250 do not benefit from the use of the Eurocode.

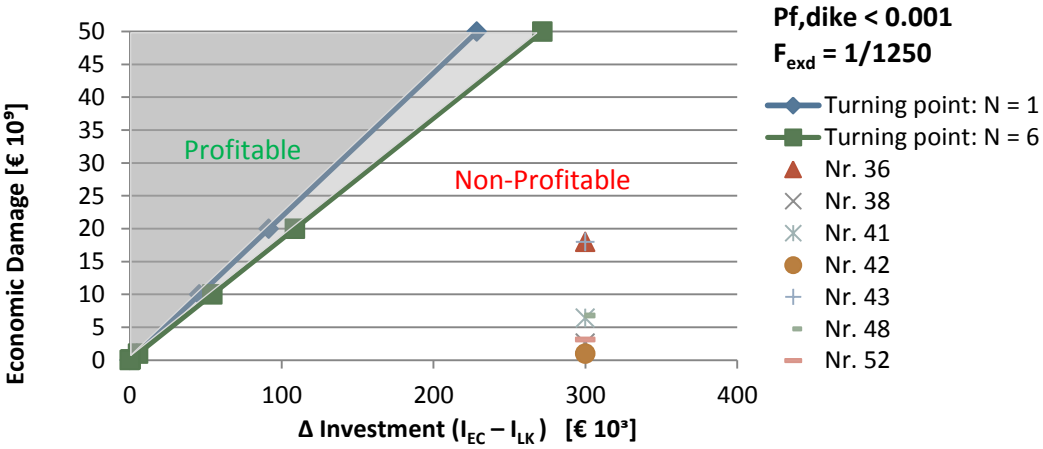


Figure 8-16: The input of a series dike rings with an exceedance frequency of 1/1250. The investment is derived from the case study in chapter 6 and the economic damage from VNK.

The same analyses are made for the frequencies of 1/2000 and 1/4000 and the resolved in the same conclusion as was given for the analysis with the frequency of 1/1250. For the frequency of 1/10000 a different conclusion was found. In Figure 8-17 the same relation as Figure 8-15 is presented, only now the exceedance frequency is set on 1/10000. Again the existing dike rings in the Netherlands (Nr. 13 and Nr. 14) with a corresponding frequency are added to the figure (again, these data follows from Table 3-4). The only problem is to generate a reliable estimation of the required investment. The reliabilities of the Eurocode and LK (for an exceedance frequency of 1/10000) are not far from apart, which implies that the required investment will be lower than is assumed in Figure 8-16. For this reason, the exceedance frequency of 1/10000 the investment for a single lock is set on 100.000 €.

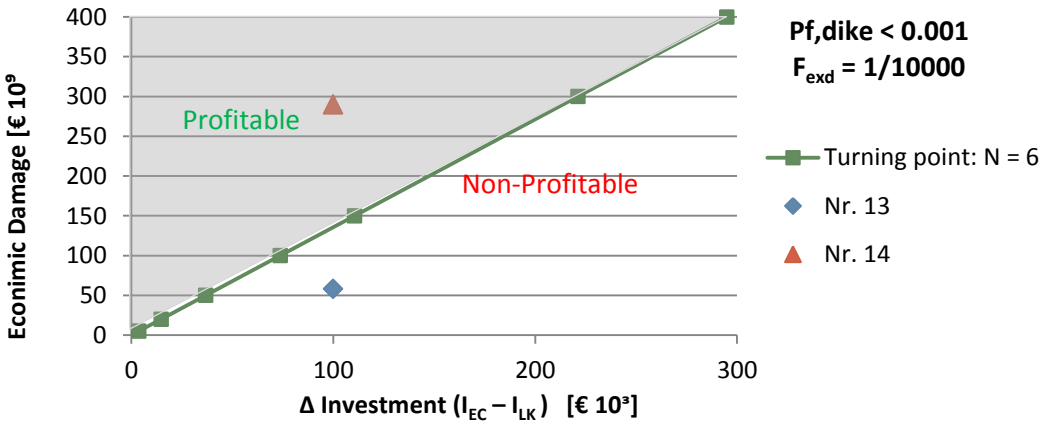


Figure 8-17: The input of a series dike rings with an exceedance frequency of 1/10000. The investment is estimated on 100.000 € and the economic damage generated by VNK.

From economic perspective, Figure 8-17 suggests that dike ring number 13 (South-Holland) should use the Eurocode. At the other hand, dike ring number 14 (North-Holland) should not apply the Eurocode according the analysis, while both areas are entitled with an exceedance frequency of 1/10000. So, for these navigation locks it is doubtful to conclude of the Eurocode is applicable or not.

These conclusions are actually a bit short sighted. The analyses consists a couple of simplifications which cannot be neglected. The applied correlations are not accurately derived. They utilize roughly estimated influence coefficients. However, any changes in the correlations will not result in large differences in the findings. Also the consequence (economic damage) is a very simplified value. The consequence of flooding includes some more extensive analysis than just a single value. The location in the dike-ring where failure occurs and the dimension of time should have a more explicit role in the analyses. The economic damage approach makes an analysis like these more approachable. Furthermore, the estimated investments are not reliable values. For more reliable conclusions, a more accurate research is required.

8.8 Alternative Measures

An increase in strength and stability of the lock is not the only measure which decreases the risk. There are more measures possible. Actually, the hardest way to decrease the risk of flooding is by increasing the strength and stability of the lock. In section 8.2 it is already mentioned that the failure mechanism strength and stability has a much lower influence than the failure mechanism closing and overflow.

So the question arises, why invest so much to increase the strength and stability of the lock, if the same result can be achieved with a smaller investment in the failure mechanism overflow or closing. In order to understand how large these differences are, table 8.8 and 8.9 presents the required reliability of the different mechanisms to create the same reliability for the dike ring.

Table 8-8: Required reliability of the alternative measures. The situation is sketched for a dike ring with an exceedance frequency of 1/1250. The number of lock which is involved is 6 and the dike section has a failure probability which equals to the exceedance frequency.

	Initial Reliability	*Measures on…* Strength	Closing	Overflow
$\beta_{Strength/Stability}$	4,314	**5,200**	4,314	4,314
$\beta_{Closing}$	3,775	3,775	**3,793**	3,775
$\beta_{Overflow}$	3,168	3,156	3,156	**3,168**
$\beta_{Dike\ Ring}$	2,864	2,869	2,869	2,869

Table 8-9: Required failure probability of the alternative measures. The same situation is adopted as is explained in table 8.8.

[1/year]	Initial Failure Probability	*Measures on…* Strength	Closing	Overflow
$Pf_{Strength/Stability}$	8,00E-06	**9,96E-08**	8,00E-06	8,00E-06
$Pf_{Closing}$	8,00E-05	8,00E-05	**7,45E-05**	8,00E-05
$Pf_{Overflow}$	8,00E-04	8,00E-04	8,00E-04	**7,67E-04**
$Pf_{Dike\ Ring}$	2,09E-03	2,06E-03	2,06E-03	2,06E-03

In table 8.9 is noticed that the failure probability regarding strength and stability decreases with a magnitude of almost 100. In contrast, the failure probability regarding overflow decrease with a magnitude of approximately 1,04. This measure seems like a quite small investment. Overflow is especially depending on the water level. A measure to satisfy a lower failure probability against overflow will be accomplished by increasing the height of the door. To give an indication about the magnitude of such an increase, a small example calculation is carried out. For this example the water level from Driel are used.

$h_{1/1250}$ (F_{exd} = 1/1250 = 8,00E-04) = +12.50 NAP [m]
$h_{1/1304}$ (F_{exd} = 1/1304 = 7,67E-04) = +12.53 NAP [m]

So, an increase of the doors with 3 cm resolves in the same decrease in risk in case the Eurocode is applied for designing the structure. However, the increase of 3 cm of the doors seems a much cheaper measure as designing the lock on basis of the Eurocode. It is worth to consider these alternatives since it can save a lot of money in the design.

8.9 Conclusion

The case study resulted in some interesting observations regarding the sensitivity of the variables in a flood defense system. According the analysis, the use of the Eurocode resulted in a small decrease in risk. The extent in which the risk decreases depends on a couple of parameters. Remarkable was the limited influence of the dike and the number of locks. Concerning the dikes, in Figure 8-12 is shown that for a relatively high failure probability of the dike, the decrease in risk becomes constant. This concludes that the dike section theoretically seen could be excluded from the analysis.

Besides the dike sections, the number of locks play a limited role in the analysis. If more lock are involved, the larger the decrease will be. However, the investment increases as well, causing to total costs almost unchanged. This is observed in Figure 8-15, which indicates the situation in which the use of the Eurocode is a success. The boundary between success and no success is expressed with a boundary line. The boundary lines for a single lock and 6 locks are not far from apart. But, there is a small difference which cannot be neglected.

Finally, several dike-rings in the Netherlands were assessed. For these dike-rings the economic influence of applying the Eurocode is researched. If the analyses resulted in as increase of the total costs, it would be, economically seen, recommended to use the Eurocode for the design. These analyses are expressed in Figure 8-16 and Figure 8-17. It concludes that the Eurocode is not of any benefit for the exceedance frequencies 1/1250, 1/2000 and 1/4000. At the other hand, the conclusion for the exceedance frequency of 1/10000 is not the same for every dike ring.

However, the analyses consists a couple of simplifications which cannot be neglected. The applied correlations are not derived accurately. Those are based on roughly estimated influence coefficients. Also the consequence (economic damage) is a very simplified value. The consequence of flooding includes some more extensive analysis than just a single value. The location in the dike-ring where failure occurs and the dimension of time should have a more explicit role in the analyses. The economic damage approach makes an analysis like this more approachable. Furthermore, the estimated investments are not accurate values as well, these depends too much on the considered locks. For more reliable conclusions, a more accurate research is required.

But why should one increase the failure mechanism (strength and stability) with the lowest failure probability. It is researched in what extend the other failure mechanisms should be improved to reach the same improvement of the dike-ring in case the Eurocode is applied. By improving the failure mechanism overflow (increasing the height of the gates) a remarkable observation was found. An increase of the doors with 3 cm resolves in the same decrease in risk in case the Eurocode is applied for designing the structure. However, the increase of 3 cm of the doors seems a much cheaper measure as designing the lock on basis of the Eurocode. It is worth to consider these alternatives since it can save a lot of money in the design.

9 Conclusions and Recommendations

9.1 Conclusions

The conclusions of this study are presented in the form of a comprehensive answer to the main research questions that were presented in Chapter 5.

What are the additional costs for a lock design based on the Eurocode, and to what extent do the structural elements influence that?

The consequences of the application of the Eurocode are noticeable. The entire project costs increases by approximately 0.5% to 0.9%. This is based on a lock with a corresponding exceedance frequency of 1/1250. These results are specifically valid for the discussed case study, but it is expected the extent of the project does not influence the results significantly. What does matter is the exceedance frequency of the lock (and corresponding dike section). For high frequencies, the load partial factor for water level differences decreases. This results in lower additional investment costs for the gate.

In the case study, different comparisons are made. The combination NEN/LK is applied as representative combination. This combination is compared with the second and third reliability class of the Eurocode, and with a combination of both. In this last combination, it is assumed that only the elements that are related to the flood defense system are designed based on the highest available class. The other elements are assigned to reliability class 2. This combination lowers the increase approximately 0.29%.

In the comparison the elements are separately discussed. The two elements that are affected significantly are the lock gate and the lock head. For the lock head, the increase is caused by a different conception of the shears stress in the Eurocode. The minimal shear capacity of the concrete is much lower. For the lock gate, the partial factor for water level increase significantly in relation to the reliability class. This last observation makes the two other case studies even more interesting.

To what extent is the current determination method for the partial factor for water level differences applicable and how should the method being adjusted?

In Chapter 7 the results of the FORM analyses are compared with the detailed semi-probabilistic method described in Section 4.6. The objective of the analyses was to see if the influence coefficient is applied correctly. In Equation 7.17 a requirement was introduced, by which the correctness of the influence coefficient could be derived. If the requirement was satisfied, the deflection of the reliability index (between the

FORM analyses (Method II) and the semi-probabilistic method (Method I)) should be smaller than 0,5. Besides the requirement, the deflection could be calculated as well.

From the comparison made Section 7.5 a larger influence coefficient was found than currently is applied (-0,7) in order to achieve an accurate partial factor. However the requirement from Eq. 7.17 is met in every scenario. This means the deflection of the reliability index should be smaller than 0.5 and the standard influence coefficient (-0,7) can be considered to be correct.

In addition, the actual deflection of the reliability can be calculated as well (to verify if the requirement (Eq. 7.17) is correct). The deflection should be calculated for the partial factors for the loads and for the strength. If a deflection is found for the loads in one way, the deflection of the strength will be the other way around. Only by combining these to deflection the final deflection can be calculated.

In this case study, this deflection is only calculated concerning the load part (partial factor for water level differences). In some situations the deflection was slightly larger than 0,5 (approximately 0.6). It seems realistic to assume that this deflection will be restored to 0.5 (or smaller) by the combining the deflection on the strength side. To be sure, the deflection of the strength should be calculated as well. This requires more research. But basically, it seems that the influence coefficient is used in the correct way for the partial factor for water level differences

Concerning the simplified method, another conclusion is found. This conclusion can be considered separately from the analyses to the influence coefficient. In the simplified method for determining the partial factor for water level differences, a number of variables are generalized, including the characteristic water level difference. In Section 6.4 is explained that this standardized method results in different reliability indexes for the same partial factor (see Figure 7-17). So, in order to design in an accurate way, it would certainly not be recommended to apply one partial factor for different the water level difference.

Are the reliability classes represented in the Eurocode applicable for hydraulic structures? This can be rewritten into the question: Are the failure systems between HS and buildings comparable? Is the investment (from LK to EC) lower than the decrease in risk?

This would be a difficult question to answer. The failure systems are of course not the same. However, from the case study, it follows that the failure probability of dikes has a limited influence on the decrease in risk. From a relatively high failure probability, the decrease of the risk became constant. From this observation, it can be concluded that the dike section could be excluded from the analyses. What is basically left are a number of locks.

The use of the Eurocode is of economic benefit if the required investment is lower is than the decrease in risk. The situation in which the profit is zero (so the investment and decrease in risk are equal) is called the turning point. This turning point decides whether the use of the Eurocode is a success (the amount of profit does not really matter to answer the research question). From the analysis, it follows that the effect of the number of locks on the turning point is limited. Instead, the investment and the

consequence of flooding have a large effect on the turning point. Unfortunately, these two parameters are difficult to derive.

From the VNK project in the Netherlands some dike rings are assess for their economic damage. These data is used in the analysis, in order to generate some conclusions about the applicability of the Eurocode for the flood defense system. The dike-rings with high exceedance frequencies (1/1250, 1/2000 and 1/4000) seems to have an unfavorable benefit from the use of the Eurocode. These areas are linked with relatively low economic damages. For the exceedance frequency of 1/10000 it is doubtful what the effect of the use of to Eurocode does. However, the analyses consists a couple of simplifications which cannot be neglected. For the correlations, consequences (economic damage) and the investment a reliable value is missing. These values are based on estimations. For more reliable conclusions, a more accurate research is required.

But why should one increase the failure mechanism (strength and stability) with the lowest failure probability. It is researched in what extend the other failure mechanisms should be improved to reach the same improvement of the dike-ring in case the Eurocode is applied. By improving the failure mechanism overflow (increasing the height of the gates) a remarkable observation was found. An increase of the doors with 3 cm resolves in the same decrease in risk in case the Eurocode is applied for designing the structure. However, the increase of 3 cm of the doors seems a much cheaper measure as designing the lock on basis of the Eurocode. It is worth to consider these alternatives since it can save a lot of money in the design.

Final Conclusion:
The law prescribes to use the most stringent standard for lock that is part of the Flood Defense System. This is a given that is difficult to avoid. However, what would help are some adjustments in the standards to make them more applicable for locks. The correct partial factors should be applied and the right classification of the dike rings should be. There are optimizations possible for the determination method of the partial factor for hydraulic loads. Concerning the reliability classifications, from an economic point of view, it seems that the dike-rings with higher exceedance frequencies (1/1250, 1/2000, 1/4000) do not benefit from the use of the Eurocode. This is in contrast to the lower exceedance frequency (1/10000). This concludes that it would not be recommended to use the Eurocode for the dike-rings with higher exceedance frequencies. There are also other measures which could be applied. , it is found that an increase of the gate with 3 cm results in the same improvement of the dike-ring in case the gate is designed based on the Eurocode. This seems to be a less expensive measure and should not be neglected.

9.2 Recommendations

Based on the conclusions a list of recommendations is presented here.

- The minimal shear capacity for concrete differs significantly from the capacity one knows from the NEN. This causes some problems for thick walls. Solutions should be found how to deal with these situations.

- In the case study in Chapter 6 is proposed the design each element of a structure with a separate reliability class. Normally if a reliability class is chosen, the whole structure should be designed based on that reliability class. There are strong arguments for choosing to design the structural elements (of a lock that is part of the flood defense system) separately. Elements such as the gates, stability of the lock head and protection against piping should be designed in the highest category (reliability class three). The other elements should be designed in reliability class two. With this recommendation, unnecessary costs are avoided.

- It would be recommended to start a research to the deflection of the reliability regarding the strength partial factor. With this deflection the total deflection (strength + load) of the reliability index can be compute, and the influence coefficient can be evaluated.

- Research should be done including the Gumbel parameter affecting the influence coefficient and the partial factor (especially the scale parameter). If the influence is small, it could be considered to standardize this parameter in order to make the determination method more accessible for desinging. Hence, the inner water level is still included in the determination method.

- In section 4.6.3 is noticed that the number of data point which are applied in the determination of the Gumbel parameters, have a large influence on the magnitude of these parameters. Subsequently they also influence the density function and exceedance frequency of the outer water level, thus the load partial factor. This influence should be researched, in order to find a proper way to deal with the data points.

- The material correction factor that is applied in the LK is based on the reliability of the material partial factor of the NEN. This correction factor should be adjusted to the material reliability represented by the Eurocode.

- The current applied length effect for hydraulic structures that are part of the flood defense system should be studied. The increase of 10% seems to be too conservative. From the calculations corresponds the 10% to serial system of 8 locks. This could be an option for an optimization.

- By further research, the currently used reference period of 10 years could be optimized. The applied reference period has a significant influence on the partial factor for water level differences. Possible adjustments of the reference period could increase the partial factor for water level differences.

- The current load partial factor for water level differences is applied in the water pressure. With this manner, the design water pressure not expressed as it truly working water pressure. For vertically loaded elements this can be a problem. Research may lead to an optimization.

- In addition on the case study from chapter 8, an actual dike ring could be applied to analyze. Up till now, the case study applies only not existing dike systems. It just showed the relations and importance of the elements involved. With a concrete example, the consequence can be defined specifically as well as the applied correlations in the dike ring and the investments. If such an analysis is carried out for several dike rings and are registered, the results could probably conclude something about the applicability of the Eurocode for the flood defense system.

- It would be recommended to consider alternative measures to improve the dike ring. For example the mechanism “overflow”: by slightly increasing the height of the gates, the same improvement of the dike-ring is reached. It is worth to consider these alternatives since it can save a lot of money in the design.

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