Future of weir Linne

Main report

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Preface

First of all, I would like to thank my graduation committee for their guidance. Special thanks to Ad van der Toorn for his daily guidance and critics, to Bas Jonkman for his support in helping me with the structure of the report and to Rene Braam for his support into the calculations of the concrete gate. I would also like to specially thank Henk Verkerk for introducing me into the Meuse system and helping me with information about the Meuse corridor. Furthermore, I would like to thank my fellow students of the Delft University of Technology for the discussions and joyful moments. I would like to thank my parents, friends, and other family for supporting me during my entire study. At least I would especially thank ir. Henry Tuin about the possibility to use his graduation report "new canalization of the Nederrijn and Lek" (Tuin, 2013). This report gave me the al lot of information about weirs. Also the approach and structure of Henry's report were very clear. I have tried to write this report according to the same approach and structure. Again many thanks.

Contents

List of symbols

Latin letters

A	support reaction
A_c	dross sectional area of concrete
A_s	amount of longitudinal reinforcements steel
$A_{s,min}$	minimum reinforcing area
$B_{Poirée}$	discharge width of the Poirée part
B_{Meuse}	width of the Meuse
C	Chezy coefficient
C_i	coefficient
C_α	correction factor for angle of the gate
C_r	correction factor for rounding of the crest
C_{df}	drowned flow reduction factor
C_e	effective discharge coefficient
c_{nom}	nominal concrete cover
c_{min}	minimum concrete cover
d	internal lever arm
E_{cm}	secant modulus of elasticity of concrete
E_s	design value of modulus of elasticity of reinforcing steel
F	fetch [m]
$\vec{F}$	resulting force
F_{wave}	wave load
f_{ctm}	mean value of axial tensile strength of concrete
f_{ck}	characteristic compressive cylinder strength of concrete at 28 days
f_{yk}	characteristic yield strength of reinforcement
F_s	flow induced load
G	vertical load due to the upstream water level
g	gravitational acceleration
H	energy height
H_s	significant wave height
h_{up}	upstream water level
h_{down}	downstream water level
h_{cr}	crest height
$h_{cr,Stoney}$	crest height of the Stoney gates
i_b	bottom gradient
k	factor
L	length of the weir gate
M	bending moment
M_{Ed}	design value of the applied internal bending moment
M_{cr}	cracking moment
M_u	moment of failure
m	discharge coefficient
m_{pw}	discharge coefficient of a perfect weir
m_{iw}	discharge coefficient of a imperfect weir
N_S	load in the reinforcement steel
N_C	load in the compression zone
O	area
P	force due to air pressure
p_a	air pressure

p_w	water pressure
Q	discharge
ΔQ	resulting discharge (difference between inflow and outflow)
Q_1	horizontal load due to the upstream water level
Q_2	horizontal load due to the downstream water level
q	discharge per unit width
r	radius
S	self weight
S_{sub}	submerged self weight
T	Tension
t	thickness
u	velocity
V_{ol}	volume
V_{Ed}	design value of the applied shear force
$V_{Rd,c}$	shear resistance of concrete
v_{min}	minimum shear stress
W	width between two supports
w	weight
w_{max}	maximum crack width
x_L	contact length
x_u	height of the compression zone
$x_{i,max}$	maximum height of the compression zone

Greek letters

α	angle
α_e	ratio
α_c	fullness factor
β	distance factor
γ_C	partial factors for concrete
γ_S	partial factors for reinforcing steel
δ	increment
ε_{cu2}	ultimate strain of concrete
θ	angle
ρ	density of water (= 1000 kg/m^3)
ρ_c	density of concrete (= 2650 kg/m^3)
ρ_l	reinforcement ratio for longitudinal reinforcement
σ_{cp}	compressive stress in the concrete from axial load
σ_s	steel stress
ϕ	diameter reinforcing steel
ϕ_{sheet}	angle
Ψ	factors defining representative values of variable actions

Summary

The Meuse in the Netherlands is since the 1920s dammed by 7 weirs, which are located near Borgharen, Linne, Roermond, Belfeld, Sambeek, Grave and Lith. The weirs regulate the water levels in the canal sections (=stuwpanden) to provide sufficient depth for navigation. The management of the weirs is regulated by water level measurements at Maastricht St. Pieter (formerly Borgharen). The weirs dam the river for discharges lower than 1200 m3/s. For higher discharges the weirs are completely opened.
Weir Linne is the oldest weir of the canalized Meuse and is part of the smallest canal section (=kanaalpand) of the Meuse. The weir consists just like the weirs of Roermond, Belfeld and Sambeek of two parts, a rough discharge regulating Poirée part and a accurate regulating Stoney part. Weir Linne maintains a water level of 20.80 m +NAP in the upper canal section. The canal section forms a navigable interchange between the Juliana Kanaal, Lateraal Kanaal, Kanaal Wessem-Nederweert and canal section Roermond.

Problem statement

The weirs in the Dutch part of the Meuse have been constructed for a period between 80 and 100 years. The weirs are approaching the end of their lifetime in the period between 2020 and 2030. In this situation is not spoken about design lifetime because in the period when the weirs were constructed the design lifetime was not defined. Based on maintenance reports and inspections it is concluded that the weirs are not able to fulfill their functions properly, they need to be replaced or upgraded. The main reasons for replacement or upgrading are technical and functional aging and changing legislations.
According to Rijkswaterstaat the weir of Linne has ended its lifespan in the year 2030. Aging of materials and changes in the ARBO legislations are issues of the weir. This conclusions is the result of inspections and analyzes over several years. Besides aging of materials also aging in functionality contributes to the decrease of the weir quality. The functionality of the weirs has been changed in time due to changing conditions, in the RINK project called "new risks". These functional changes can have their causes in changes of design assumptions, maintenance, usability and controllability or law and regulations. The weir of Linne has to deal with the risks of changing discharges, ASR degradation, operability and the weir does not meet the ARBO regulations.

Approach

This graduation report is focused on the problems of weir of Linne. The aim of this thesis research is to improve the situation of weir Linne for the future situation by replacing of upgrading the weir. The weir should meet the requirements based on discharge and navigational purposes in the future. This report is made up at four scale levels, the problem is approached from a high scale level to lower scale levels. The levels are made up according to the elementary design cycle.

The first scale level concerns the canalized Meuse in the Netherlands. Because canal section Linne is relatively small It is investigated if it is possible to remove weir Linne instead of renewing or upgrading of the weir. The second scale level focuses on canal section Linne. In this section the effect of the Maasplassen with respect to the water level changes of the canal section is investigated for high and low discharges. Also the effect of the water level changes to the weir management is investigated to obtain a clear view about the reaction time of weir operations with regard to opening and closure of the weir. The third design level concerns the construction of weir Linne. It is considered if according to the RINK reports the current weir construction should be renewed or upgraded. The chapter proceeds with a rough analysis about the required width and retaining height of the weir to obtain if the discharge capacity of the current weir is sufficient for the future. It is decided to upgrade the Poirée part of the weir, the last part of this design level concerns the determination of a new gate type for the Poirée part of the weir. The lowest design level is focusing on the design of the new weir gate. The gate is designed in high strength concrete. The dimensions of

the gate elements are determined and the gate is checked to the requirements according to Eurocode 2. The last section of the lowest design level concerns the way of placing and connecting the new gate to the current construction.

Results

The result of the first design level is that abolishing canal section Linne by combining the canal section with the upstream or downstream canal sections is not advisable. Weir and weir section Linne should be maintained in the future. This hold also for the other canal sections of the Meuse corridor except for canal section Lith. Because this canal section is the most downstream section in the Meuse canalization and far away from Linne, interventions to this canal section would have no effect on weir and canal section Linne. However the result holds for the current layout of the Meuse corridor. If a new layout should be proposed for the canalization of the Meuse it should be analyzed if weir Linne should be maintained or removed.

The Maasplassen have a damping effect on the water level changes in canal section Linne. The damping effect is the effect of the relation between the changing discharge with respect to the water level changes in the canal section. Due to the Maasplassen the water level changes develop more slowly compared to a situation without the Maasplassen. This is the result of the storage capacity of the Maasplassen. The damping effect of the Maasplassen is most preferable and effective during low discharges. Navigational dept is maintained for a longer period because the water level decrease proceeds slowly due to the connected lakes. The discharge accuracy of the weir is not sufficient for the small discharges in the future, to maintain the navigable depth during low discharges the discharge accuracy should be increased. The damping has less effect on extreme high discharges compared to low discharges. Because the canal section is relatively small the water level rise will be relatively quick regardless of the damping effect of the Maasplassen. An increase of the storage capacity of the Maasplassen has a minor influence on this effect. However, the banks and retention areas are omitted in the analysis of the effects of the Maasplassen to the water level changes in canal section Linne. The results are not exact however, relative to each other the variants do represent a clear view about the effect of the Maasplassen to the water level changes.
The opening procedure shorter than 8 hours is reliable after high discharges are measured at measure point Maastricht St. Pieter. Investigations on running times of flood waves on the Meuse have been done are not valid any more. Due to the Grensmaasproject running times for food waves will change.

It is not advisable according to the RINK (=Risico Inventarisatie Natte Kunstwerken) report to replace the total weir of Linne at the moment. It is advised to monitor the condition of the weir until the period 2030 - 2035 and to take after this period a final decision. After this period a decision should be made to replace the or maintain the current weir for a longer period. In this report is concluded to maintain the current weir construction instead constructing a new weir. Due to over dimensioning twhe concrete super construction is a robust construction. The upgrade of the current weir will be done by the removal of the Poirée gates and constructing a new gate on the old Poirée part. The Poirée gate does not meet the requirements according to the ARBO legislations and the gate is not able to be remote controlled. The new type of weir that is chose to replace the old Poirée gates is the inflatable flap gate. The flap gate is able to retain a the required water level, able to close the old Poirée part without large reconstruction of the weir. The gate fulfills (when properly designed) the ARBO legislations and is able to be remote controlled.

The flap gate is designed in concrete class C90/105, this is a high strength concrete. High strength concrete is a durable construction material and requires less maintenance compared to steel constructions, especially in wet environments. The gate will be supported three air filled bladders, the bladders are placed next to each other. The concrete construction meets the requirements according to Eurocode 2. The checks of reinforcement, cracks and moment of failure are done for

bending moments and shear resulting of loads perpendicular to the gate. Torsion due to for example failure of bladders is not taken into account. Checks to the concrete gate with regard to moments, cracks and failure due to torsion moments are also not taken into account.
The gate will be placed as a prefabricated element. The element contains the gate, bladder, chain and supporting bottom plate. The plate is on upstream side placed at the rotation notches of the former Poirée gates and on the downstream side to the weir bottom. The gate could be further optimized, optimization could be done by for example reducing of the gate weight and curving the gate to increase the discharge capacity.

1 Introduction

1.1 Introduction to the Meuse

The Meuse is a river in the western part of Europe. The Meuse arises in France, about 100 km north of Dijon. The Meuse passes besides France also Belgium and the Netherlands where the water is discharged into the North Sea. The total length of the Meuse from source to the North Sea is about 935 km. The Meuse in France arises at a height of 409 m + NAP on the Plateau of Langres. The river moves in northerly direction through a valley in the Plateau of Lotharingen, at this location the width of the Meuse becomes narrow. The French part of the Meuse has two important tributaries, the Chiers and the Semois. Nearby the Ardennes Plateau the Meuse enters Belgian territory. From Sedan the French part of the Meuse is partly navigable for recreational boating and Class I vessels.

The Meuse enters Belgium at a height of 98m +NAP. The Meuse moves in northerly direction to the city of Namur and further in easterly direction until Liege. At this point Meuse has a height of 60 m + NAP. After Liege the Meuse moves in northerly direction to the Dutch border. The Meuse is totally canalized in Belgium and suitable for navigation. The canalization is completed by means of 12 weirs. From the Belgian-French border the river suitable is for vessels of the IV class and from Namur for vessels of the Va class.

The Meuse enters the Netherlands near Eijsden, the water level of the Meuse has at this location a height of 57.68m +NAP. The river flows across the province of Limburg via Maastricht, Roermond and Venlo to Cuijk. Between Borgharen and Maasbracht the Meuse is the natural border between Belgium and the Netherlands. At Cuijk the Meuse leaves Limburg and enters the provinces of North Brabant and Gelderland. The Meuse moves from Cuijk along Ravenstein, Maasbommel and 's-Hertogenbosch to Heusden. In this section the river forms the natural border between the provinces of North Brabant and Gelderland. From Heusen the Meuse turns into the Bergsche Maas. The Bergsche Maas directs in easterly direction until Raamsdonksveer. At Raamsdonksveer the Bergsche Maas becomes the Ammer and moves in easterly direction until Moerdijk. At this location the Ammer confluences with the Nieuwe Merwede. The Nieuwe Merwede is connected to the Waal and has the purpose to discharge a part of the Waal water. At the confluence the water flows through the Hollands Diep and the Haringvliet to the North Sea. The described trajectory of the discharge from Eijsden to the North Sea has a length of about 315 km. At part of this trajectory vessel up to class Vb are allowed and parts allow vessels up to class Va. It is important to notice that the part of the Meuse which in Dutch is called *"the Meuse"*, is the river part from Eijsden until Heusden. When in this report is spoken about the Meuse, the section between Eijsden and Heusden is meant.

The Roer, the Niers and the Swalm are important Meuse tributaries in the Netherlands. The tributaries discharge water on the Meuse. The watersheds of these tributaries are mainly located in Germany, the watershed of the Roer is the largest (Breukel, Silva, van Vuuren, Botterweg, & Venema, De Maas - Verleden, heden en toekomst, 1992).

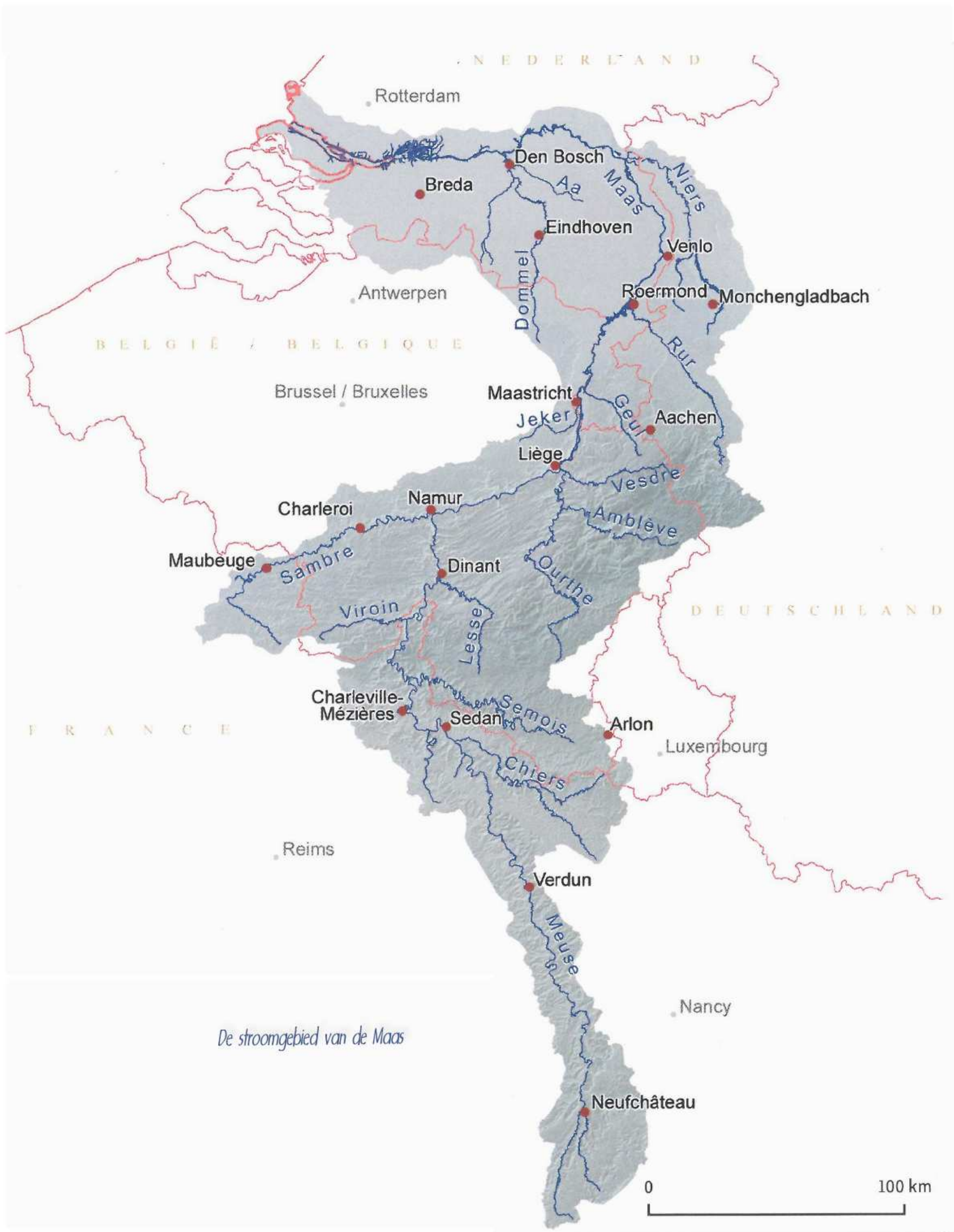


Figure 1 - Flow of the Meuse (source: Maascommisie)

1.2 History of the canalized Meuse in the Netherlands

The demand to optimize the Dutch rivers for navigational purposes increased around 1900. Led by the industry and trading companies the pressure on the government to optimize the rivers increased rapidly, in particular for the Meuse. New techniques were developed for large scale coal winning from the mines in southern Limburg. The coals should be transported to the industrialized areas in the western part of the Netherlands. Transport by ship was the cheapest solution. However during large periods of the year the water depth of the Meuse was too shallow for navigation. It was decided to canalize the Meuse. The canalization and normalization of the Meuse was done for two purposes:

- To make the Meuse suitable for large vessels
- Better discharge to prevent floods at the downstream regions of the Meuse.

In the beginning of the 20the century new techniques were developed in hydraulic engineering. However most Dutch engineers had no experience with these new techniques. Due to a lack of experience a lot discussion strarted about the type of weirs that should be used for the Meuse canalization. An proven but old fashioned design like the Poirée weir or a modern weir like a weir with Stoney gates? A combination of both principle was chose and resulted in a weir that consisted partly of Poirée gates and partly of Stoney gates.

It lasted until 1919 before the canalization was started. Not only the discussion about the weir type but also the difficult negotiations with Belgium to canalize of the Border and the start of the First World War delayed the process. In 1919 the Dutch government decided to start with the canalization of the Meuse on Dutch territory. The canalization was started with the construction of the weirs near Linne, Roermond, Belfeld, Sambeek and Grave.

Afther the First World War no agreement with Belgium could be reached about the canalization of the Border Meuse. The Dutch government decided to construct a channel parallel to the Border Meuse on own territory, the Julianakanaal. To maintain the water depth of the Julianakanaal a weir near Borgharen was constructed. The weir of Lith is the last constructed weir of the Meuse canalization. After normalization between Grave and Blauswsluis weir Lith was constructed. The weir is constructed to obtain sufficient navigational depth between Grave and Lith during low discharges of the Meuse. The locations of the weirs are represented in Figure 2. The canalization of the Meuse was divided into four parts.

- Part 1: Canalization of the Meuse from Maasbracht to Grave by means of five weirs near Linne (1921), Roermond (1921), Belfeld (1924), Sambeek (1925) and Grave (1926).
- Part 2: Construction of the Julianakanaal between Borgharen and Maasbracht (1926 to 1936) and construction of a weir near Borgharen (1928).
- Part 3: Canalization of the Meuse from Grave to Lith by constructing a weir nearby Lith (1936).
- Part 4: Normalization of the Meuse between Grave and Blauwesluis to obtain a better discharge though this part of the river.

From (Berger & Mugie, 1994) and (Heezik, 2008)

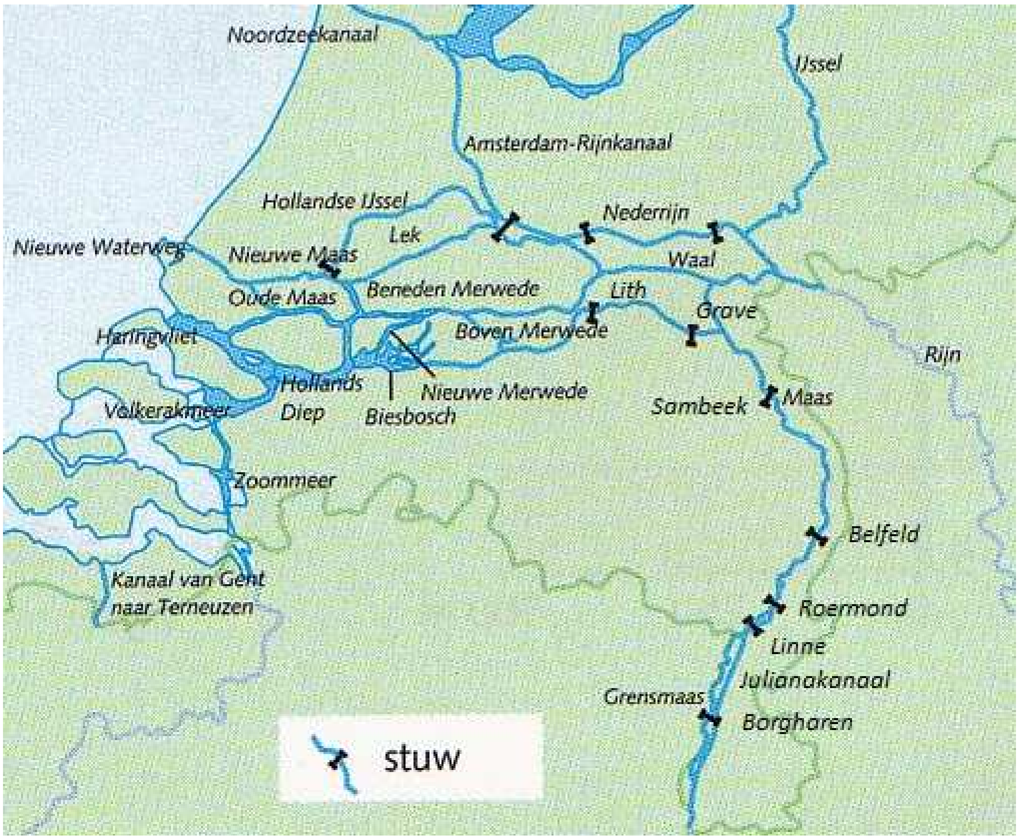


Figure 2 - Locations of the 7 weirs of the Meuse (source: Rijkswaterstaat)

1.3 Problem description

In the Dutch part of the Meuse seven weirs have been constructed in the period between 1921 and 1936. They are located near Borgharen, Linne, Roermond, Belfeld, Sambeek, Grave and Lith. The main function of the weirs is to create sufficient water depth for navigational purposes.
The weirs have been constructed for a period between 80 and 100 years, just like a lot of other hydraulic structures. The weirs are approaching the end of their lifespan in the period between 2020 and 2030. Based on maintenance reports and inspections it is concluded that the weirs are not able to fulfill their technical functions properly so they need to be replaced. The main reasons for replacement or upgrading are technical and functional aging. Also increasing discharges due to climate change is an important aspect. 5 Of the 7 Meuse weirs require manual operation. When discharges due to climate change increase, weirs should be opened more often so more frequent manual operation of the weirs is necessary. However Rijkwaterstaat would like to operate more frequent from one central point instead of on several locations. Operation at one location should improve the traffic flow on the Meuse and better anticipation to high discharges.

Irrespectively of the Meuse corridor or the division of the Meuse in channel sections (=kanaalpanden), the weirs of Linne and Borgharen are important for navigation at the Meuse. Without the weirs between Borgharen and Grave no navigation between Belgium and the Waal is possible. Without the weirs of Linne and Borgharend also no connection on Dutch territory is possible between the Meuse and the Kempische Kanalen. Weir Linne provides navigation at the Meuse in the direction of the channels in the province of North Brabant by means of Kanaal Wessem-Nederweerd. The weir of Linne has been built in 1921 and is therefore the oldest weir in the Meuse. Because the Kanaal Wessem Nederweert, canal section Roemond and to the Lateraalkanaal are connected to the channel section of Linne, the weir plays an important role at this navigational interchange.

According to Rijkswaterstaat has the weir of Linne ended its lifespan in the year 2030. Aging of materials is one of the minor issues of the weir. This conclusions is the result of inspections and analyzes over several years. Besides aging of materials also aging in functionality contributes to the decrease of the weir quality. The functionality of the weirs has been changed in time due to changing conditions, in the RINK
project called "new risks". These functional changes can have their causes in changes of design assumptions, maintenance, usability and controllability or law and regulations The weir of Linne has to deal with the risks of changing discharges, ASR degradation, operability and changes in ARBO regulations.

1.4 Goal

The goal of this thesis research is make a design to improve the situation of weir Linne for the future situation. The weir should meet the requirements based on discharge and navigational purposes. Four design levels between the Meuse system to gate design are districted to achieve the main goal of this report. Every level contains additional research questions that are answered in the associated chapters. The results of the chapters are taken to the next chapter. In section 1.4.1 to 1.4.4 are the research questions of the four design levels represented.

1.4.1 Chapter 2: System analysis

The weirs in the Meuse becoming outdated which results in impropriate functioning. For the purpose of management and maintenance an alternative layout with less weirs could be a option. It should be investigated if and which weirs could be removed, if weir Linne is one of these weirs and the effect of the removal of one of the weirs with respect to weir Linne. The questions to be answer in chapter 2 are:

- Is a different configuration of weirs feasable?
- Can canal section Linne be retained?
- What is the effect of the removal of weirs on canal section Linne?

The result of this chapter is used in chapter 3. If weir Linne should be retained, this result is used in chapter 3. If weir Linne should not be retained, it should be investigated which weir of the Meuse corridor should be selected to complete this graduation report.

1.4.2 Chapter 3: Canal section Linne

The behavior of the weir with respect to high and low future discharges has to be investigated. The behavior of the weir management is related to the response of the gravel lakes located in canal section Linne. The effect of the lakes on the behavior of the weir to discharges and the accuracy of the new situation has to be investigated. The questions to be answer in chapter 3 are:

- In which way should the discharge regulation of weir Linne behave to high and low discharges?
- What is the effect of the lakes in canal section Linne on the discharge regulation of the weir?

The result of this chapter that will be used in the next chapter is the maximum required time to open the weir. The maximum time depends on the velocity of the water level rise in the canal section. Also information about the accuracy is could be used in chapter 4. The results of this chapter will be used in the next chapter to obtain the requirements of the behavior for a new or upgraded weir.

1.4.3 Chapter 4: Weir complex Linne

The weir of Linne approaches the end of its lifespan in the period between 2020 and 2030. It should be investigated if it is necessary to renew the total weir or that adjustments to the Poirée or Stoney parts of the current weir construction also satisfy for the future. Wherever adjustment and renewing of the weir is necessary, investigation to an new type of weir gate is necessary to improve the situation of weir Linne. The questions to be answer in chapter 4 are:

- Should the entire weir be renewed or parts of the weir?
- Which type of gate type should be used for the new situation

The result of this chapter is a decision if the weir should be renewed or upgraded. Also a problematic element for a upgraded weir of a element of a new weir should be determined. This element will be worked out in the last chapter.

1.4.4 Chapter 5: Gate design

The gate has to be designed, weir gates are usually designed in steel. Due to the introduction of new and improved materials are other designs possible. Several polymer lock and weir gates have been designed, however not frequently applied. Lock gates out of high strength concrete have been applied at lock 124 at IJburg. However a concrete weir gate has never been constructed, in chapter 5 is the possible application of a concrete flap gate investigated: The questions to be answer in chapter 5 are:

- Is it possible to design a weir gate of high strength concrete?
- How should the gates be placed with respect to the discharge regime of the Meuse?

1.5 Reading guide and approach

1.5.1 Scale levels

This report is made up from a high scale level to lower scale levels. Scale levels are important for civil engineers because. Applying of scale levels is mainly expressed by refining the design on several scales. For the macro scale level the goal, functions and design are determined. On lower scale levels is mainly focused to the feasibility of the components and elements. The design becomes more detailed for lower scale levels which results in smaller uncertainties in the design. On the other hand possibilities to intervene decrease on lower scale levels.
In civil engineering are for practical reasons often three scale levels distinguished however more scale levels are often used. The three scale levels are:

- Macro level (system)
- Meso level (component)
- Micro level (element)

In this report four scale levels are defined, the levels are:

- Canalized Meuse in the Netherlands (system)
- Canal section Linne (sub-system)
- Weir complex Linne (component)
- Weir gate (element)

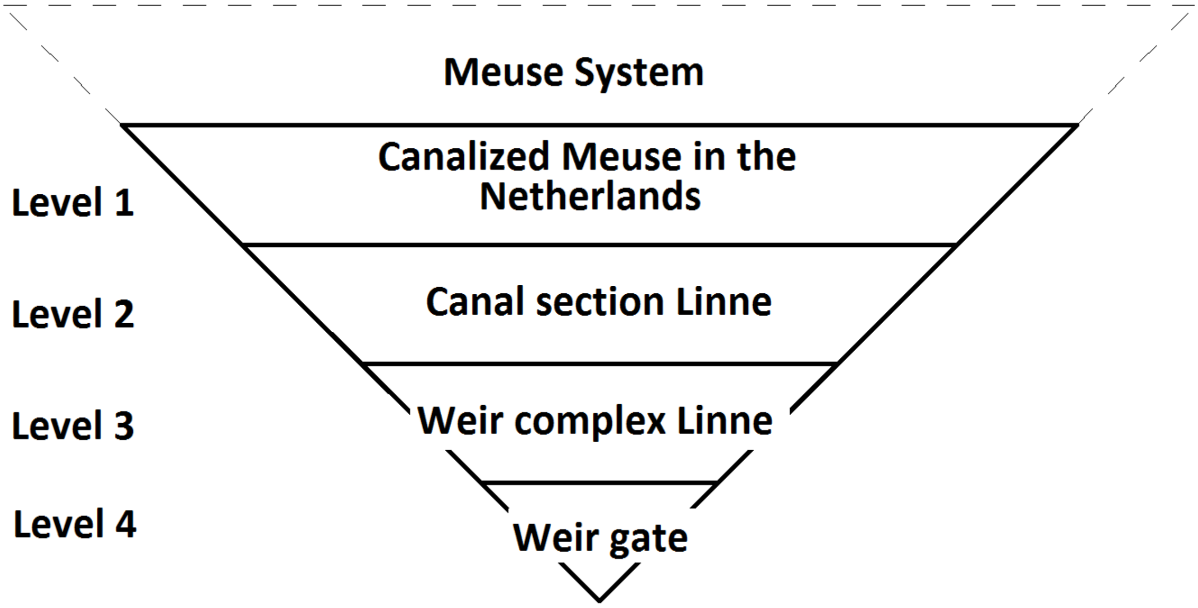


Figure 3 - Levels of the report

1.5.2 Elementary design cycle

The design method used in this report is the elementary design cycle, the design cycle is presented in Figure 4. The design cycle is applied to the four scale levels as presented in section 1.5.1. The elementary design cycle consists of five steps, the steps are:

- *Analysis*
  In the analysis step is information given about the system. Out of the information are the criteria developed for the next design step of the elementary design cycle
- *Synthesis*
  At the synthesis are possible solutions developed
- *Simulation*
  In the simulation step is the behavior tested of the possible solutions (variants) developed in the synthesis step. Also the value and costs of the developed constructions are taken into account.
- *Evaluation*
  In the evaluation step is the order of variants determined from best to worst
- *Decision*
  In the decision step is determined which variant will be elaborated on the next design level.

The design cycle includes two feedback possibilities. If is decided that a proposed solution is not acceptable, the solution should be adapted. The design cycle is again applied to the adapted variant. Two feedback possibilities are included in the design cycle, the possibilities are:

- Feedback by synthesis is based on the development of new and better variants. Due to a higher level of knowlege about the design task better variants could be designed.
- Feedback by analysis is based on the development of new and better design criteria. Just as for the feedback by synthesis better criteria could be obtained due to a higher level of knowelge about the subject.

Every chapter is roughly made up accoriding to the elementyary desing cycle. The chapters start with an analysis of the desing level. Information of the design level, the functions, requirements and boundaries of the specified design level are determined. The chapters continue by determining severeal variants at the scale level. The variants are simulated according to simple moddels to determine the behaviour of the variants. The chapters end with a evaluation of the simulated variants. The decisions step is not included in the chapters.

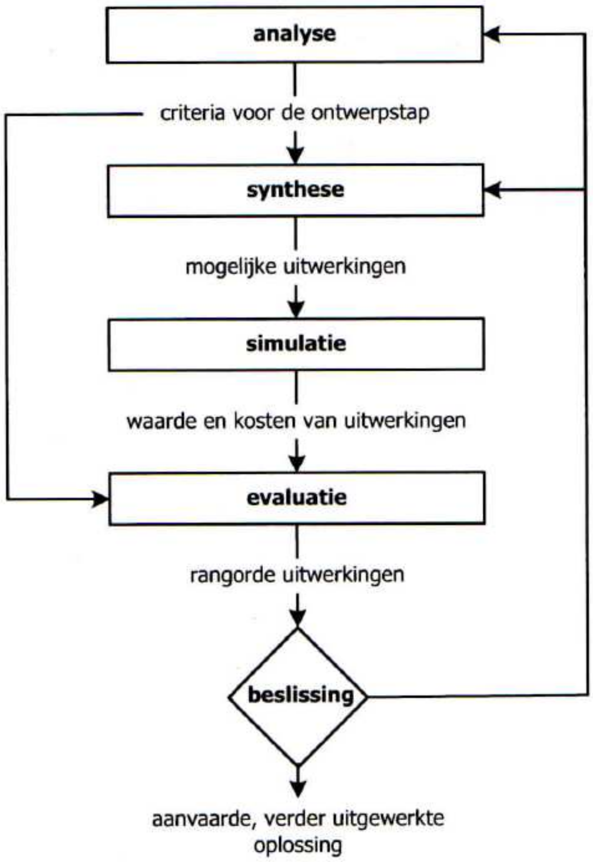


Figure 4 - Elementary design cycle (souce: Tu Delft, lecture notes CT1061)

1.5.3 Multi criteria analysis

A multi criteria analysis (MCA) is a valuable tool to aid decision making where there is a choice to be made between competing options. A MCA supports the comparison of different variants based on a set of criteria. The different developed variants are compared to the set of criteria. Per criteria a value is given to the variant. Different ways of valuing are possible, possibilities are from -- to ++ or from 1 to 10. The difficulty of these ways of rating is the clarification between two consecutive values, like 5 and 6 or + and ++. It is not always clear why for example 5 point are given and not 6 points.

To avoid indistinctness in rating of the variants another way of valuing is chosen. At the MCA analysis in this report the worst variant is valued with 1 point the second worst with 2 point up to the best variant.The MCA is a valuable tool but is able to give a misleading view in the choice of variants. Attention should to be given to the determination of characteristic criteria for the variants. It is important to ensure a certain balance between the criteria. To determine a clear balance between the criteria, the criteria are clustered by determining the characteristics and the importance of the criteria with respect to the design task. The balance is made by placing the criteria in a matrix and compare the variants with each other. The most important criteria is esteemed with a 1 and the criteria of minor important with a 0, in this manner it is indicated which of the two criteria is more important. The balancing is done by comparing all the criteria each other. At the end the importance and weight factor of the criteria relative to each other is determined. An example of the determination of the weight factors is presented in Table 1.

Table 1 - Determination of weight factors

	Criteria A	Criteria B	Criteria C	Criteria D	Criteria E	Criteria F	Criteria G	Total	Weight factor
Criteria A		1	1	1	1	1	1	6	0.29
Criteria B	0		0	1	1	1	1	4	0.19
Criteria C	0	1		1	1	1	1	5	0.24
Criteria D	0	0	0		1	1	1	3	0.15
Criteria E	0	0	0	0		1	0	1	0.05
Criteria F	0	0	0	0	0		0	0	0
Criteria G	0	0	0	0	1	1		2	0.10
								21	1

An example of a MCA as it will used in this report is presented in Table 2. The rated values per criteria are multiplied with the weight factors. By multiplying the rates by weight factors a balanced score per variant is obtained. Based on the MCA a conclusion is made about the variant which will used.

Table 2 - Multi Criteria Analysis

Aspects	Weight factor	Variant 1	Variant 2	Variant 3
Criteria A	0.29	2	1	3
Criteria B	0.19	2	1	3
Criteria C	0.24	2	3	1
Criteria D	0.15	3	2	1
Criteria E	0.05	2	1	3
Criteria G	0.10	3	1	2
Score excl. factor		14	9	13
Score incl. factor		2.29	1.65	2.18

2 System level - analysis of the canalized Meuse

The second chapter contains the analysis of the Meuse corridor and the importance of the canal sections. In this report is the canal section used for the Dutch word 'kanaalpand' For the purpose of management and maintenance is in this chapter investigated if and which weirs could be removed and canal section combined. This will result in an a conclusion which weirs could possibly removed and if weir Linne could be retained. Also the effect of the removal of one of the weirs with respect to weir Linne is taken into account because changes of water levels to adjacent canal sections could affect the load distribution on weir Linne.

The process is started in section 2.1 with a description of the Meuse system, the main aspects in the description are navigation and discharge. The main functions of the Meuse system are determined in section 2.2. The area analysis and the main functions of the system results in a set of requirements which are represented in section 2.3. The system boundaries are represented in section 2.4 and the boundary conditions of the system are represented in section 2.5. The synthesis of an alternative division is presented in section 2.6. The section describes the possibility of removal of weirs and combining of canal sections in the new configuration of the Meuse corridor. For every canal are the effects analyzed for the removal of a specific weir. The canal sections are modeled for the most negative situation, a situation with no discharge. The water levels are therefore modeled with a horizontal water level. The conclusion of the design level is given in section 2.7.

2.1 System description

The system description provides an overview of the canal section on the Dutch part of the Meuse corridor. A schematization an a cross section of the Meuse corridor are represented to obtain an overview of the system layout.

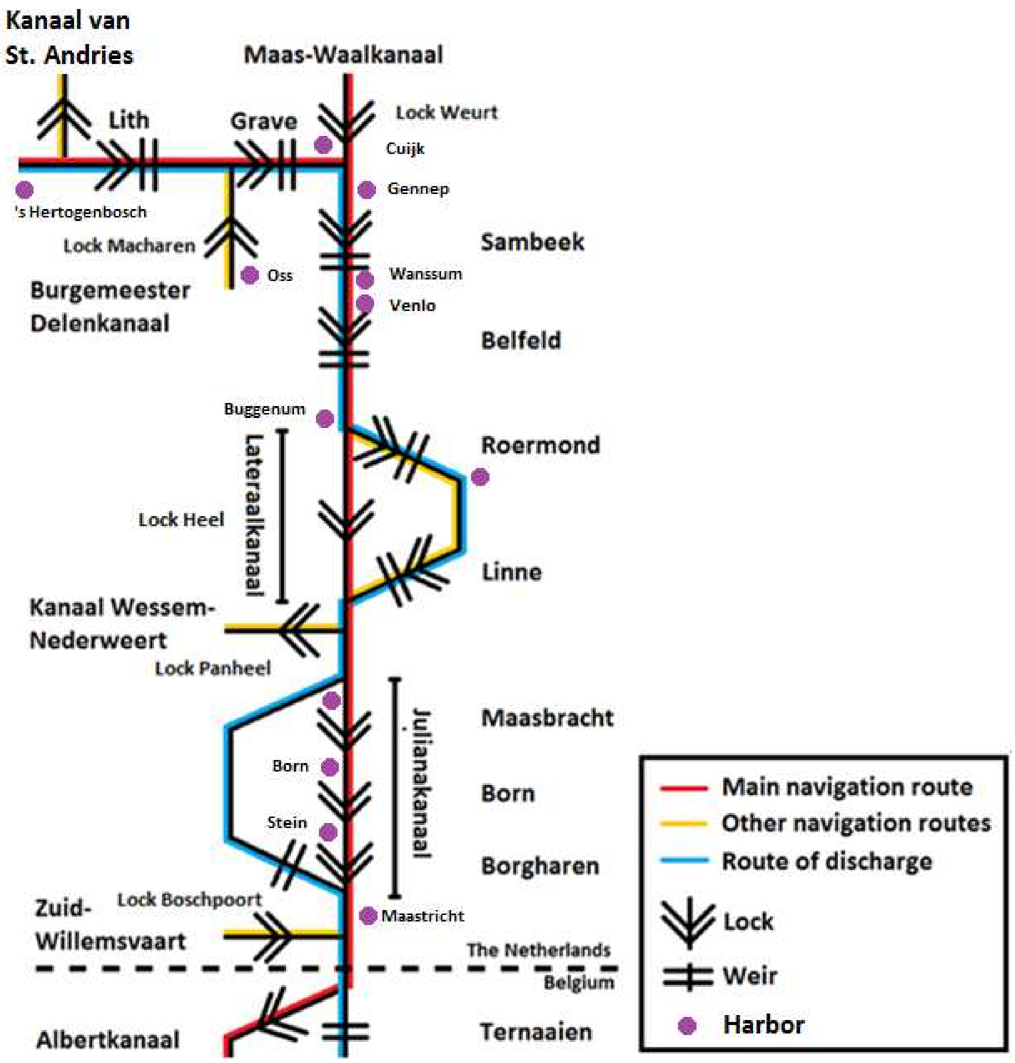


Figure 5- Schematization of the Meuse, representing the navigational route, route of the discharge and weirs and main lock

2.1.1 Sections

The Dutch part of the Meuse is divided in 7 canal sections (in this report is the term canal section used if the Dutch word stuwpand is meant). The 7 Canal sections are controlled by weirs 7 weirs. Besides the 7 canal sections the Meuse contains for navigational purposes 2 channel sections located in the Julianakanaal (in this report is the term channel section used if the Dutch word kanaalpand is meant). The channel sections are controlled by two locks, lock Born and lock Maasbracht. The weirs and locks maintain the required water level in the canal and channel sections. The weirs and locks are located on the downstream side of the canal sections.

The weirs are located near:

- Borgharen
- Linne
- Roermond
- Belfeld
- Sambeek
- Grave
- Lith

Two canal sections are located at the Juliana Canal. For these two canal sections are no weirs used because the Juliana Canal has no discharge function. A small amount of water is discharges by the canal to maintain the water levels of the two canal sections. The controlling locks are located near:

- Born
- Maasbracht
-

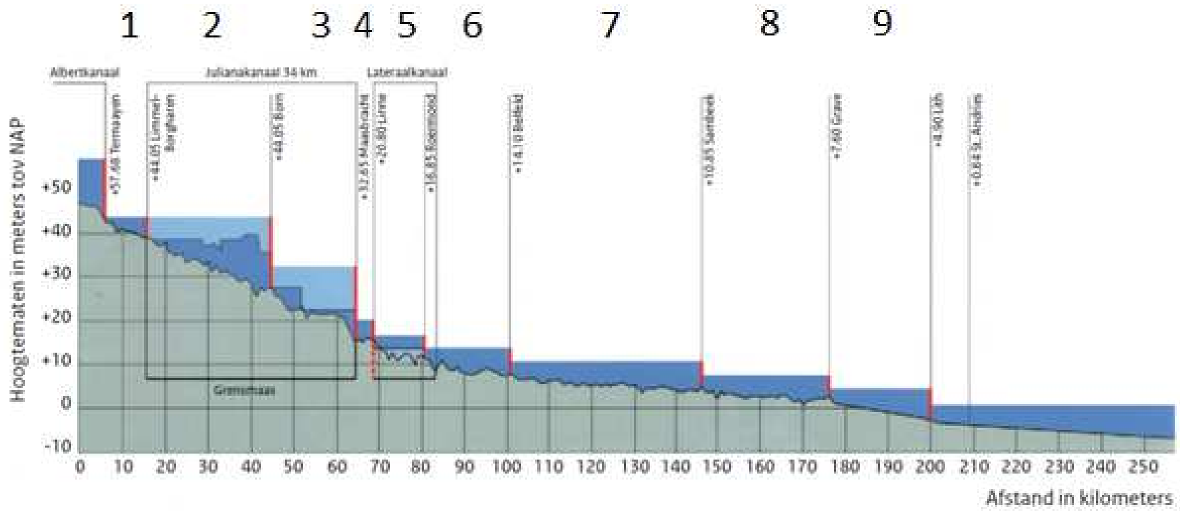


Figure 6 - Representation of the Meuse corridor and corresponding water levels (source: Rijkswaterstaat)

Table 3 represents the mean width and the bottom gradients for the canal sections as represented in Figure 6. The sections presented in Figure 6 are the sections between weir Lixhe (Belgium) and weir Lith. In the table is also the mean width and bottom gradient of the Border Meuse included however this section is not part of the canalized Meuse corridor. The Border Meuse is included to obtain a better view of the progress of the width and bottom gradient. As expected is the mean width increasing for a decreasing bottom gradient.

Table 3 - Mean width and bottom gradients for the canal and channel sections of the Dutch part of the Meuse (source: Rijkswaterstaat)

Section	Section Type	Mean Width [m]	Bottom gradient [‰]
Lixhe (België) - Borgharen	river (1)	120	0.34
Borgharen - Born	channel (2)	100	0
Born - Maasbracht	channel (3)	100	0
Maasbracht - Linne	river (4)	120	0.3
Borharen - Linne (Borde Meuse)	*river*	*120*	*0.3*
Linne - Roermond	river (5)	120	0.15
Roermond - Belfeld	river (6)	140	0.08
Belfeld - Sambeek	river (7)	140	0.10
Sambeek - Grave	river (8)	160	0.12
Grave - Lith	river (9)	160	0.10

2.2 Functions

The Meuse system has several functions. The functions are divided into main functions and secondary functions. The main functions are the most important tasks of the system to perform in a right way. The system of the Meuse contains two main functions, the main functions are:

- *Transport of water*
  Water has to be transported from the upstream boundary to the downstream boundary. The system should be able to transport a certain amount of water in a safe manner according to safety norms.
- *Navigation*

The system should be able to provide navigational transport from the upstream to the downstream boundary. To enable navigation must the waterway have sufficient depth and width.

The secondary functions are the functions that are of less importance for the functioning of the system. Secondary functions of the system are for example drinking water, energy generation of industry (Berger & Mugie, 1994). However in this chapter are the secondary functions of no importance. The secondary functions become more important in lower levels of design.

2.2.1 Discharge

The discharge of the Meuse fluctuates during the year. The Meuse is a typical rain-fed-river, discharges in the winter months (December to March) could differ more than 1000 times from the summer discharges (June to October). The discharge originates mainly out of rain water from the Ardennes. During the winter months more rain fall is expected than during the summer months. In extreme dry or wet years, this effect is increased. The mean annual discharge of the Meuse is 230 m^3/s. The annual discharge is not evenly distributed over the year. The mean discharge during the winter months is about 480 m^3/s and in the summer months about 89 m^3/s (Bruggeman, et al., 2013). Most water is entering the Netherlands near Eijsden. However, the water discharged by the Dutch part of Meuse is not completely originated from the Ardennes. Small streams and tributaries in the Netherlands discharge water to the Meuse. These stream and tributaries together contain a mean discharge of more than 50 m^3/s. The most important are:

- Voer
- Jeker
- Geul
- Roer
- Swalm
- Groote Molenbeek
- Niers
- Graafse Raam
- Dieze (/ Dommel / Aa)

2.2.1.1 Low discharges

Extreme low discharges are discharges smaller than 30 m^3/s. Low discharge occur mainly between July and September. It is expected that extreme low discharges in the future decrease between 6 and 18 m^3/s (Bruggeman, et al., 2013). It is also expected that these discharges become more frequent. During extreme low discharges the Meuse cannot fulfill at its functions like navigation and usages of water for different activities. Problems with low discharges occur mainly upstream of Roermond. At Roermond the Meuse tributary the Roer discharges water at the Meuse. The Roer guarantees a constant minimum discharge of at least 10 m^3/s.

2.2.1.2 High discharges

The current design discharge of the Meuse is 3800 m^3/s. However, this discharge is never measured, the highest discharge ever measured is 3039 m^3/s. It is expected that due to climate change winter discharges of the Meuse in the become higher. Design discharge between 4000 and 4600 m^3/s are expected according to the KNMI'14 scenarios (Bruggeman, et al., 2013). It has to be mentioned that the design discharge of the current flood protection is 3275 m^3/s in Limburg and 3800 m^3/s downstream of Boxmeer. The current flood protection has a required probability of exceedance of 1/250 up to 1/1250 for lower parts of the Meuse (Waterwet, 2009). For increasing design discharges in the future should the flood protection be adapted to the new design discharge.

2.2.2 Navigation

2.2.2.1 Navigation route

The Meuse is an important navigational transportation route in the Netherland. Yearly more than 25.200 vessels transport more than 18.5 million tons per year over the Meuse (Provincie Limburg, 2008). The Meuse connects the Netherlands to the international network of waterways. The river is connected to the Trans European Network of waterways by means of the Meuseroute (Dutch: Maasroute). The Meuseroute contains two branches, namely:

- North - south branch (from Ternaaien to Heumen, vertical red line in Figure 5)
- East - west branch (from Heumen to Hedel, horizontal red line in Figure 5)

Together with the Albert Canal, the Scheldt-Rhine Canal and the Waal mentioned as "de Grote Ruit" (Provincie Limburg, Netwerkanlyse vaarwegen en binnenhavens, 2008). An overview of the Big Rhombus is represented in Figure 7.

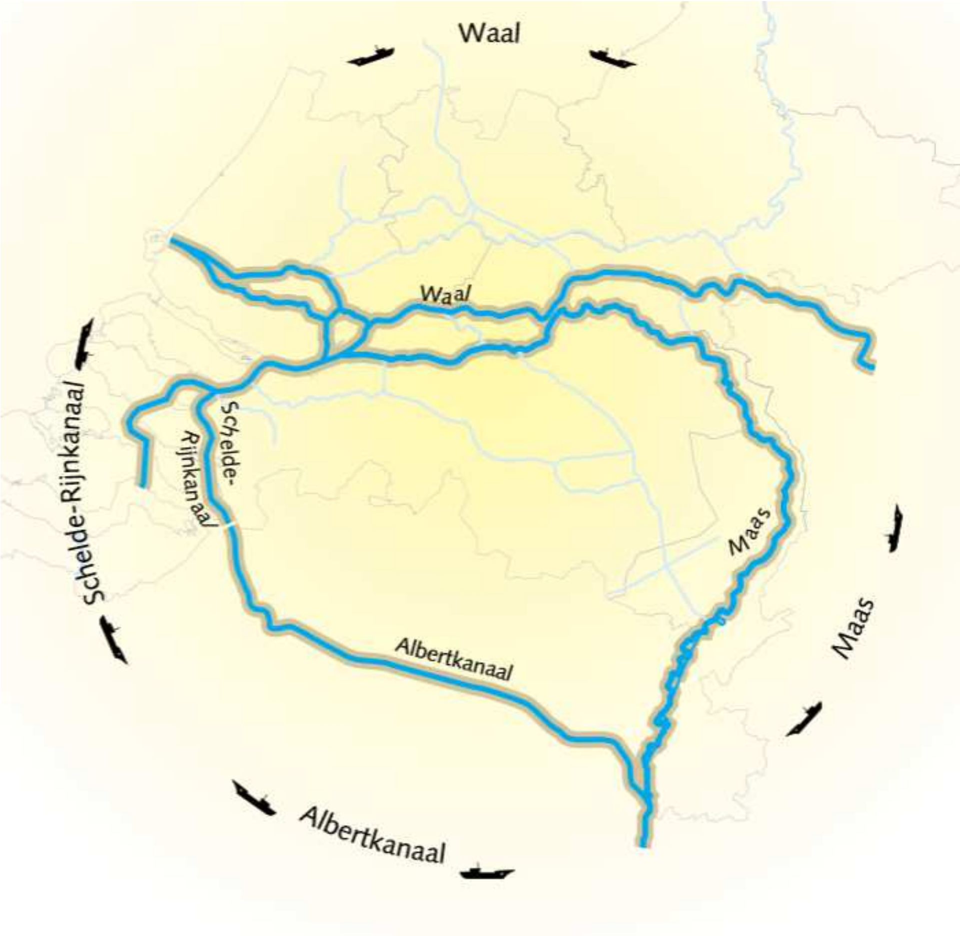


Figure 7 - Grote Ruit including Meuse, Waal, Albertkanaal and Schelde-Rijnkanaal (source: Rijkswaterstaat)

De grote Ruit is an important transport connection between the Netherlands, Belgium, Germany and France (Rijkswaterstaat, Maas: verlenging sluiskolk bij Heel, 2014). It is important to maintain the Meuseroute for navigation. No navigation on the Dutch part of the Meuse results in more transport over the Albert Kanaal. The Netherlands will lose the economical potential to Belgium. To maintain the competitiveness of the Netherlands. The Meuseroute does not contain all parts of the Meuse. To improve the navigability two canals are included in the Meuseroute, the canals are:

- *Julianakanaal*
  The Julianakanaal is located parallel to the non canalized Border Meuse. The Border Meuse is the natural border between the Netherlands and Belgium.
- *Lateraalkanaal*

The Lateraalkanaal is located parallel to the winding river section of Roermond. The canal decreases the navigable distance. Vessels are able to pass one lock instead at Heel instead of two locks in the river section Roermond. The water level of the Lateraalkanaal is maintained by the weir of Belfeld.

The Meuse contains four connections to enter or leave the Meuseroute. The most important entries, from upstream to downstream are:

- Zuid-Willemsvaart
- Kanaal Wessem-Nederweert
  (provides access to the Kempische Kanalen in the Netherlands and Belgium)
- Maas-Waalkanaal
- Burgemeester Delenkanaal
- Kanaal van St. Andries

2.2.2.2 *Types of navigation*

The Meuse is used by transport vessels and recreational vessels. The total Meuseroute is suitable for vessels of the CEMT Va class, however from 2015 will the north - south branch be suitable for class Vb vessels. The increase of vessel class on the Meuse is important to maintain the competitiveness of the Netherlands with respect to Belgium. Not all entries to the Meuse are suitable for Vb class vessels. The Zuid-Willemsvaart and the Kanaal Wessem-Nederweert are suitable for vessels up to the CEMT II class (Provincie Limburg, Netwerkanlyse vaarwegen en binnenhavens, 2008).

2.2.3 Harbors

The Meuse contains two types of inland harbors, harbors for commercial shipping and marinas for recreational boating. The commercial harbors taken into account, the marinas are behind scoop of this report. The commercial inland harbors are situated in the provinces of Limburg and North Brabant. 9 Harbors are located in Limburg and 3 in Noord Brabant. Inland harbors have an important logistical function for the industries in the province of Limburg (Provincie Limburg, Netwerkanalyse Vaarwegen en Binnenhavens, 2008). The harbors are situated near:

- Maastricht
- Stein
- Born (important container harbor)
- Maasbracht
- Roermond
- Buggenum
- Venlo
- Wanssum
- Gennep

The harbors along the Meuse in the province of Noord Brabant are located near (Gedeputeerde Staten van Noord-Brabant, 2008):

- Cuijk
- Oss
- 's Hertogenbosch

2.3 Requirements on system level

A full list of requirements is represented in Appendix E. Only the main and most important requirements resulting from the list are presented in this section. The main requirements have the largest impact for the design solution. The requirements that have less effect to the design solution are listed in Appendix E. The requirements are:

- The north - south branch of the Meuseroute should to be suitable for vessel class CEMT Vb in relation to the width, depth and vertical clearance of the river and locks (section 2.2.2).
- Navigation on the main transport route may not be hampered by objects.

- The quay height of the harbors should be above water level (section 2.2.3)
- The flood protection along the Meuse must be high enough to prevent floods (section 2.2.1.2).

The first requirement has effect to the water level in the canal section and the water level has effect on the height of the weirs in the Meuse corridor. The second and the third requirement are related to the first requirement. For example should height of bridges not hamper navigation of Vb class vessels.

2.4 System boundaries

The system boundaries define which parts are included and which are beyond scope of the system. The boundaries of the system are defined by locks or weirs in the system. The system boundaries from upstream to downstream are:

- Weir of Ternaaien (upstream boundary)
- Maximal retaining height of lock Boschpoort
- Maximal retaining height of lock Panheel
- Maximal retaining height of lock Weurt
- Maximal retaining height of lock Macharen
- Maximal retaining height of lock Ternaaijen
- Weir of Lith (downstream boundary)

(Kanaal van St. Andries is beyond the considered boundaries of the system)

The system parts that are located inside the defined boundaries are:

- The Meuse
- Julianakanaal
- Lateraalkanaal
- Zuid-Willemsvaart (until lock Boschpoort)
- Kanaal Wessem Nederweert (until lock Panheel)
- Maas Waalkanaal (until lock Weurt)
- Burgemeester Delenkanaal (until lock Marcharen)

2.5 Boundary conditions

- The dimensions of the river and canals should be sufficient for the vessel class CEMT Vb.
- The current design discharges of the Meuse is 3800 m^3/s.

2.6 Synthesis on system level

In this chapter it is investigated whether it is possible to canalize the Meuse with less than 7 canal sections and the eventual effects to canal section Linne. The canal sections are modeled for the most negative situation for navigation, a situation with low discharge. The water levels are therefore modeled with a horizontal water level corresponding with a discharge smaller than 10 m^3/s. It is analyzed what the effect will be if two canal sections are combined. Combining canal sections result in less weirs (or locks) but also in changing water levels in the combined canal section. Changed water levels have effect on navigational depth, height of the flood protection, bridge height (class Vb vessels should be able to pass the bridges), locks and harbor quays. The three variants investigated are:

- *Variant 1 - Increasing water level*
  The water level in the combined canal section is equal to the water level of the former upstream canal section. This results in water level rises in the former downstream canal sections. The height of the downstream weir of the combined canal section should be increased to the new water level. The variant is represented in Figure 8.
- *Variant 2 - Decreasing water level*

The water level in the combined canal section is equal to the water level of the former downstream canal section. This results in water level fall in the former upstream canal sections. The height of the downstream weir of the combined canal section is sufficient however the bottom has to be lowered to maintain the navigable purpose of the canal section. This results in a increasing retaining height of the upstream weir, the variant is represented in Figure 8.

- *Variant 3 - Averaged water level*
  The water level in the combined canal section is equal to the average water level of the former up and downstream canal section. This results in water level fall in the former upstream canal sections and a water level rise in the former downstream canal sections. The height of the downstream weir of the combined canal section should be increased to the new water level. The height of the downstream weir of the combined canal section is sufficient however the bottom has to be lowered to maintain the navigable purpose of the canal section. This results in a increasing retaining height of the upstream weir, the variant is represented in Figure 8.

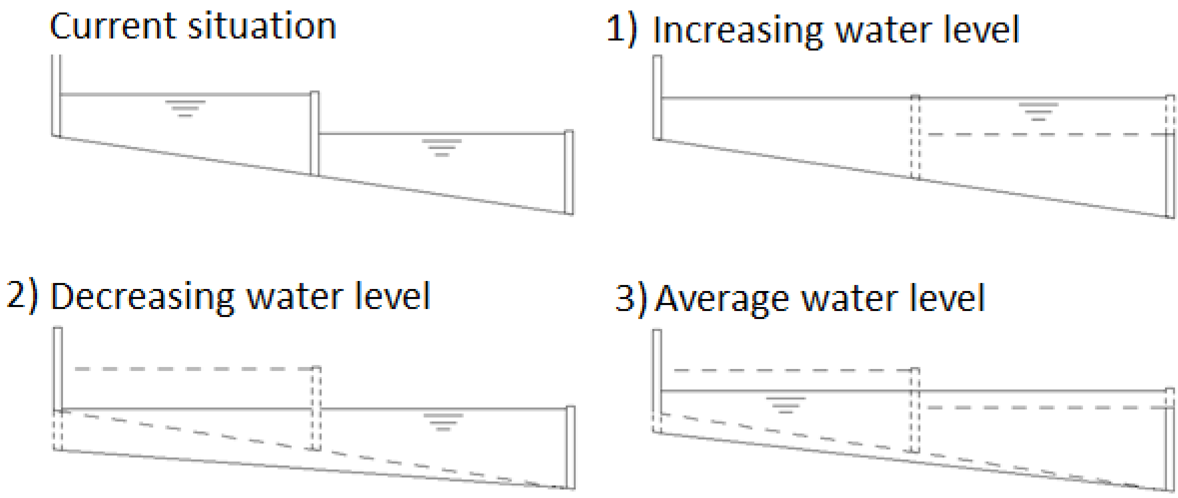


Figure 8 - Used variants for the synthesis

In Appendix F are the effects of the three variants represented. A summary is represented in Table 4. The aspects that are taken into account are:

- Navigation
  The draught of Vb class vessels is maximum 3.5 m according to the CEMT guidelines. The safety clearance between bottom and the bottom of the vessel is assumed at 0.5 m. The required navigable depth for the canal sections becomes 4 m.
- Flood protection
  The flood protection should be high enough to protect the hinterland against floods. Water level changes could have effect on the flood protection. The flood protection could become insufficient to the water levels of combined canal sections. It has to be mentioned that the current flood protection of the Meuse is sufficient for a design discharge between 3275 m3/s and 3800 m3/s. The required design discharge will increase according to the KNMI'14.
- Bridges
  Vessels of the CEMT Vb class should be able to pass Meuse bridges without problems. The height under bridges should be at least 9.10 m up to Born, southern of Born is at least 7.10 m required (Rijkswaterstaat, Waterway Guidelines, 2011). The required 7.10 m southern of Born is because Born is the most southern container harbor.
- Locks

Changes of water levels could affect the operability of locks. Locks gates can become too low to the new water levels or possibly the high and low water sides could change. This could be problematic for locks with mitre gates (= puntdeuren).

- Harbors
  Water level changes could affect the operability of harbors. If water levels rise quays could flood, if water levels fall could quay walls become instable and water depth of the harbors could become insufficient.

The red variants in Table 4 are not beneficial for the canal section. The three variants are checked on the above mentioned aspects. For every aspect is checked if adjustments are necessary, for example heightening of the flood protection or increasing the river depth for navigation. If adjustments are necessary is the aspect marked with 'no' if no adjustments are necessary with 'yes'. For the aspects: bridges, locks and harbors is the number of possible adjusted objects mentioned.

It is rated, based on the number of adjustments if combining two canal sections is a possible option. The red colored variants are not beneficial for the specific canal section. The orange colored variants are possible options however not the most beneficial. The green colored variants are reasonable possible options. Comments are represented in section 2.6.1.

Table 4 - Variant analysis for the canal sections of the Meuse

Canal section	Navigation	Flood protection	Bridges		Locks		Harbors	
	y/n	y/n	y/n	nr	y/n	nr	y/n	nr
Borgharen - Born								
• variant 1								
• variant 2	yes	no	yes		no	1	yes	
• variant 3								
Born - Maasbracht								
• variant 1	yes	n/a	no	4	yes		yes	
• variant 2	no	n/a	yes		no	1	no	1
• variant 3	no	n/a	no	4	no	2	no	1
Maasbracht - Linne								
• variant 1	yes	n/a	no	3	no	3	no	1
• variant 2	no	n/a	yes		no	1	no	1
• variant 3	no	n/a	yes		no	2	no	1
Linne - Roermond								
• variant 1	yes	no	no	1	no	1	no	1
• variant 2	no	yes	yes		no	2	no	1
• variant 3	yes	no	yes		no	3	no	2
Roermond - Belfeld								
• variant 1	yes	yes	yes		no	1	yes	
• variant 2	no	yes	yes		no	1	no	1
• variant 3	no	yes	yes		no	1	no	1
Belfeld - Sambeek								
• variant 1	yes	yes	yes		no	1	no	1
• variant 2	no	yes	yes		no	2	no	1
• variant 3	no	yes	yes		no	3	no	1
Sambeek - Grave								
• variant 1	yes	yes	no	1	no	1	no	1

• variant 2	no	yes	yes		no	1	no	2
• variant 3	no	yes	yes		no	2	no	2
Grave - Lith								
• variant 1	yes	no	no	1	no	2	no	1
• variant 2	no	yes	yes		no	2	no	2
• variant 3	no	yes	yes		no	4	no	1
Lith								
• variant 1								
• variant 2	no	yes	yes		no	2	No	1
• variant 3								

2.6.1 Combinations of canal sections

- *Borgharen - Born*

The combination between canal sections Borgharen and Born is only possible for the second variant. The current situation is during low and average discharges comparable with the second variant. The normal water level in canal section Borgharen and Born is 44.05 m + NAP and is determined by the weir at Borgharen. During low and average discharges are canal section Borgharen and Born combined. When the water level at Borgharen becomes higher than 43.65 m +NAP are the canal sections separated by lock Limmel (Rijkswaterstaat, Bedieningstijden van sluizen en bruggen). A total combination of both canal sections results in a shutdown of lock Limmel, this results in high water levels on the Julianakanaal. The Julianakanaal has no discharging function (section 2.1.1) so consequently also no flood protection During high discharges will areas along the Juliankanaal flood if no flood protection is made along the channel. However a flood protection on both sides of the channel from Limmel to Born is a expensive operation.

- *Canal sections on the Julianakanaal (Born - Maasbracht & Maasbracht Linne)*

The canal sections Born and Maasbracht are not located at the Meuse but on the Juliana Canal. The canal sections are divided by a locks instead of weirs because the Julianakanaal is used for navigation and not for water discharge. Water is discharged by the parallel located Border Meuse. A shutdown of lock Born or lock Maasbracht result in relative large water level changes in these canal sections, as can be seen in Figure 6. Without flood protection are water level changes nearly not possible. It is advisable not change the canal sections of the Julianakanaal.

- *Linne - Roermond*

Canal section Linne is relatively small compared to other sections. The section contains four locks as can be seen in Figure 5 (lock Maasbracht, lock Panheel, lock Panheel, lock Heel and lock Roermond). The canal section provides access to the Lateraalkanaal (part of canal section Belfeld), the Kempische kanalen by means of lock Panheel and canal section Roermond. Large changes in the water level of canal section Linne result in unnavigable connections.

  - Variant 1: between the new canal section Linne - Roermond and canal section Belfeld as result of a insufficient lock at Roermond. Also the recreational area of canal section Roermond is flooded due to the water level rise in the former canal section Roermond.
  - Variant 2 & 3: between between the new canal section Linne - Roermond and the Juliankanaal (lock Maasbracht) and Kanaal Wessem-Nederweert (lock Panheel). The resulting forces on the gates of lock Maasbracht and Panheel increase due to the water level decrease on the downstream side of the locks. The gates of these locks will probably not be able to withstand the increasing resulting loads.

Also the costs for reconstruction of locks, bridges and harbors in the canal section could be arguments to discourage the combination between canala section Linne and Roermond. Only the first variant could be a possible option however this option is not beneficial for canal section Roermond. The harbor and the recreational facilities of Roermond become unavailable.

- *Roermond - Belfeld*

Removing weir Roermond is possible, less hydraulic structures have to be adapted compared to the removal of other Linne. Especially the second variant could be a reasonable option in a new design of the canalized Meuse. The advantage of canal section Roermond with regard to the navigational function of the Meuse is that the section is no part of the Meuseroute. Combining canal section Roermond and Belfeld and maintaining a water level of 16.85 m +NAP is the most beneficial for navigation on the Meuse. A disadvantage is the adaption of navigable depth of canal section Roermond. The main navigable route can be maintained when weir Roermond is removed. An second advantage is the adaption of relatively less structures (bridges, locks and harbors) and the relative small height difference over weir Roemond. However the possibility of the removal of weir Roermond is investigated by students of the Hogeschool of Arnhem (de Borst, 2014). According to the report do advantages not outweigh the disadvantages and the cost of a removal of weir Roermond.

- *Belfeld - Sambeek*

Removing weir Belfeld could be a possible option for the first variant. Less hydraulic structures have to be renewed or replaced compared to the second and third variant, navigational depth is sufficient and flood protections high enough (during low discharges). The second and third variant are no good options because the navigational depth is insufficient in these variants. Increasing navigational depth by dredging is possible however both canal sections are relatively long and part of the main navigational route.
The city of Venlo is located nearby weir Belfeld. Effects of water level changes for the city of Venlo should be investigated.

- *Sambeek - Grave*

Canal section Sambeek is the largest canal section of the Dutch Meuse and part of the main navigational route. Combining canal section Sambeek with canal section Grave is not a beneficial option because canal section Grave is connected to the Maas-Waalkanaal. The Maas-Waalkanaal is an important connection between the Meuse and Waal. Most vessels use the Maas-Waalkanaal to enter or leave the Meuse. Changing water levels at the channel are not beneficial for navigation on the canal, only the second variant could be possible. However the second option results in a large combined canal section with insufficient depth upstream of Sambeek. It is advisable to maintain the canal section of Sambeek.

- *Grave - Lith*

The section constrains a connection with the Maas-Waalkanaal, an important navigable connection. Changes in the canal section Grave have effect on the Maas-Waalkanaal because the water level of the canal is maintained by weir Grave. The second and third option result in water level changes which has effect on locks and harbors in the combined canal section. The first variant could be possible because the water level in the combined canal section is equal to the water level of canal section Grave.

- *Lith*

Removal of canal section Lith could possibly be a option. By removing weir Lith becomes the canal section from Grave to the Haringvliet a free flowing river. It has to be investigated if navigation is possible from Grave to the Haringvliet without weir Lith. Also the effects on the lock in the Kanaal van St. Andries should be investigated. However most vessels that use the main navigational route enter the Meuse by the Maas-Waalkanaal. Transport at canal section Lith is less intensive which is beneficial for removal of weir Lith.

2.6.2 Combined canal sections

According to Table 4 it is undesirable to combine canal sections Borgharen, Born, Maasbracht, Linne, Sambeek or Grave. These canal sections are important for navigation on the Meuse. Bridges, harbors locks and navigational dept should be adapted in the named canal section in case of combining canal sections. The reconstruction of structures in the canal sections hinder navigation on the Meuse.

The combining of canal sections Roermond and Belfeld is a possible option according to Table 4. However the possibility of the removal of weir Roermond is investigated by students of the Hogeschool of Arnhem (de Borst, 2014). According to the report do advantages not outweigh the disadvantages and the cost of a removal of weir Roermond. It is not advisable to remove weir Roemond and according to Table 4 to combine canal section Sambeek with canal section Belfeld or Grave. The result is that canal section Belfeld should also maintained. The result of retaining weir Roemond and canal and consequently canal section Roemond has effect on the retaining height of weir Linne. The retaining height of weir Linne could be retained.

The remaining option is removing weir and canal section Lith. Navigation on this part of the Meuse could navigate to Rotterdam by the Maas-Waalkanaal and the Waal. This situation has effect to the harbor of 's-Hertogenbosch and the retaining height of weir Grave. Further investigation is necessary to obtain a clear view of the advantages and disadvantages of the effect of removing weir Lith with respect to weir Grave and the harbor of 's-Hertogenbosch.

Figure 9 represents a schematization of the canalized Meuse according to the analysis of the combining of canal sections. The water levels are bases on the 'betrekkingslijnen 1991.0'. The only possible option is the removal of canal section Lith is represented in the figure. As can be seen in the figure has the removal of canal section Lith no effect to canal section Linne.

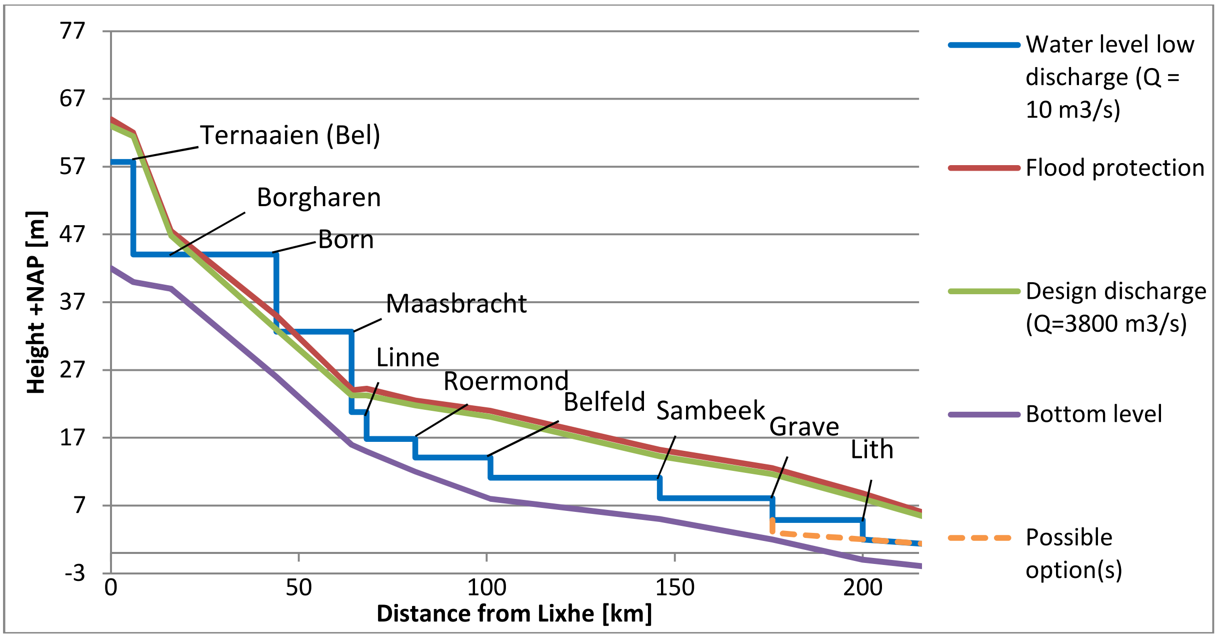


Figure 9 - Corridor Meuse in a new configuration

2.7 Conclusion

The Meuse is an important transportation route in the Netherlands. The Meuse is part of the Big Rhombus and connects the Netherlands with Belgium and France. It is for navigable purposes important to maintain navigation on the Meuse. No navigation on the Dutch part of the Meuse results in more transport over the Albert kanaal in Belgium. The Netherlands will lose the economical potential to Belgium.

To maintain navigation on the Meuse are the weirs necessary. For the purpose of management and maintenance an alternative layout with less weirs is investigated in section 2.6. According to the synthesis of the canal section should canal section Linne and consequently weir Linne be retained because the canal section fulfills an important function for navigation. The canal section forms an interchange between the Julianakanaal, the Lateraal kanaal, the Kanaal Wessem Nederweert (entry to the Kempische Kanalen) and the Meuse as can be seen in Figure 5. Combining canal section Linne with canal section Maasbracht results in extensive water level changes. Combining with canal section Roermond is not preferable because changing water levels result in navigable connections and the amount of locks, bridges and harbors in the canal section that should be reconstructed.

According to section 2.6 the canal sections upstream of weir canal section Linne should be maintained. Table 4 implies that combining canal sections Roermond and Belfeld could be a possible option. However the possibility of the removal of weir Roermond is investigated by students of the Hogeschool of Arnhem (de Borst, 2014). According to the report do advantages not outweigh the disadvantages and the cost of a removal of weir Roermond. The result is that the retaining height of weir Linne does not have to change. The load distribution on the weir does not change.

It is concluded that the weir of Lith is the only weir that could be removed, the weir no part of the main navigational route which is from Borgharen to Heumen. By removing weir Lith navigation between the Meuse and the Waal remains intact. However the effects on weir Grave and the lock in the Kanaal van St. Andries have to be investigated before a final decision is made. Discharges will increase in the future as result of climate change. The current design discharge is between 3275 m^3/s in Limburg and 3800 m^3/s downstream of Mook but will according to the KNMI'14 scenarios increase to design discharges between 4000 and 4600 m^3/s. Flood protection should be adapted to the new design discharges.

3 Canal section Linne

In the chapter 2 is concluded that canal section Linne should be retained, a weir at Linne is necessary to maintain the functions of the current weir. The discharge function should be adapted to future discharges. The accuracy of the weir with respect to low discharges is investigated in order to verify the required accuracy of the weir. The required time to open the weir is important during high discharges. It is investigated which time is required for the opening procedure of the weir to obtain sufficient discharge capacity.
The discharge regulation has effect on the water level changes in the canal section. The velocity of the water level changes is besides the discharge regulation also determined by the Maasplassen, this are lakes connected to canal section Linne. The effect of the Maasplassen on the velocity of water level changes are analyzed in this chapter.

The system of the canal section is analyzed in section 3.1. Navigation, discharge and construction of the canal section are described. The functions of the canal section are described in section 3.2. In section 3.3 the system boundaries and the boundary conditions of the system are represented. The requirements of the behavior of the canal section are described in section 3.4. Section 3.5 contains the behavior analysis water level in the canal section. Three possible variants are tested. Also the effect of increasing or decreasing the damping effect is investigated in this section. The canal section is modeled as basin with an inflow and outflow. The conclusion of the behavior of required behavior of the weir is given in section 3.6.

3.1 System description of canal section Linne

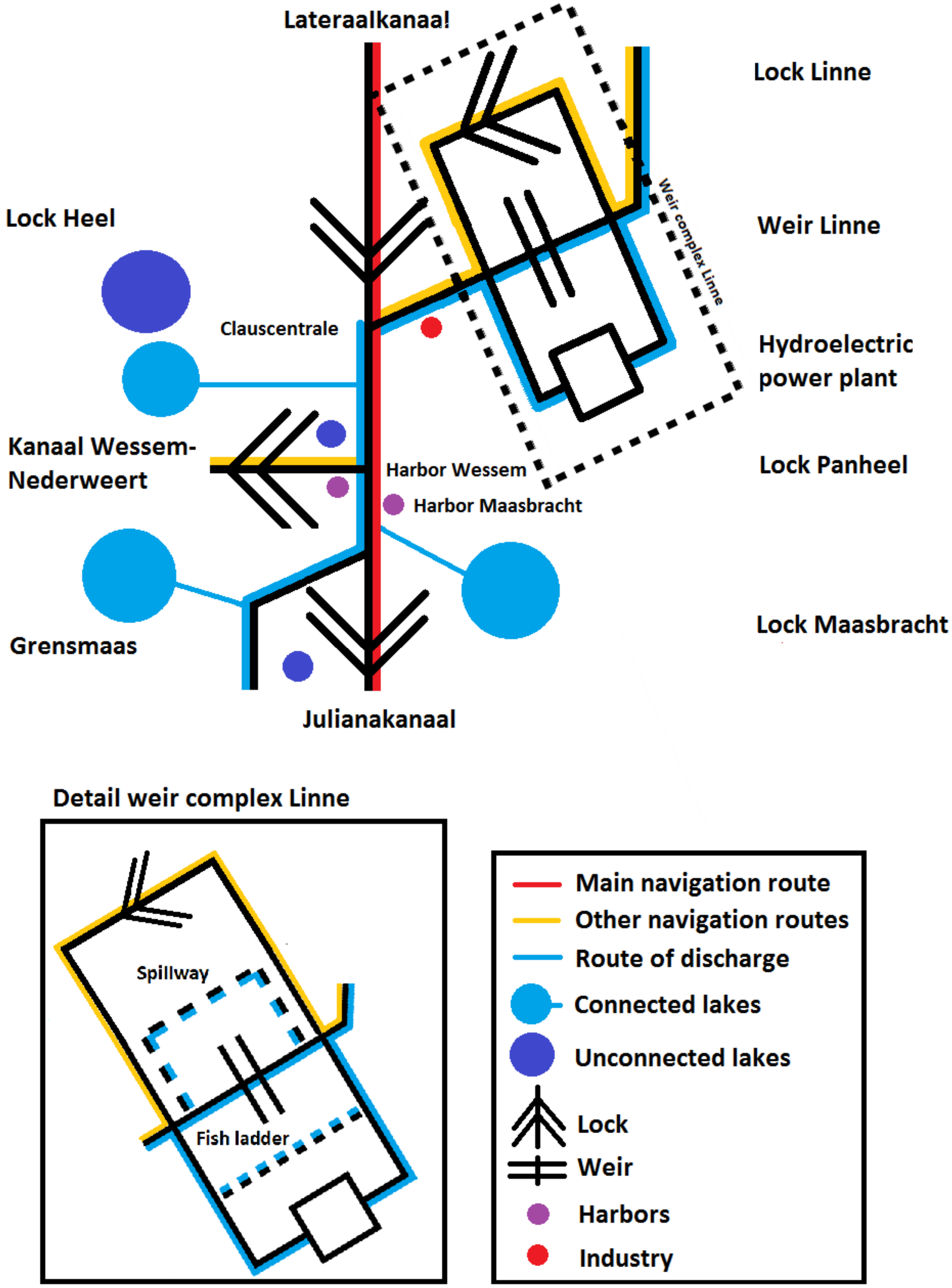


Figure 10 - Schematization of canal section Linne

3.1.1 Discharge

The discharge into the canal section is mainly provided by the Border Meuse. A minimal discharge by the Border Meuse of 10 m^3/s is required. The discharge is required for water refreshment and maintenance of the ecological aspects of the Border Meuse (Breukel, Silva, van Vuuren, Botterweg, & Venema, De Maas - Verleden, heden en toekomst, 1992). The out flowing discharge is provided by the hydroelectric power plant, fish ladder and the weir. More information about the weir and hydroelectric power plan is described in section 3.1.2.1 and section 3.1.2.2.

3.1.1.1 Duration of flood waves

Based on theoretical considerations a relation between is determined top discharges and the duration of the corresponding flood waves. The duration of the flood wave depends on the discharge capacity and the storage capacity of the river section. The discharge capacity is the accelerating part because the roughness of the riverbed relatively low due to high water levels. The storage capacity is the delaying part because the roughness of the flood planes are relatively large. In Table 5 the duration of flood waves are represented for several discharges (van der Made, 1967). It has to be mentioned that analysis is done in 1967, long before the Grensmaas project was started. In Figure 11 are the water levels before and after the flood waves of 1993 and 1995 represented. It can be seen that the days before the flood wave result in increasing water levels.

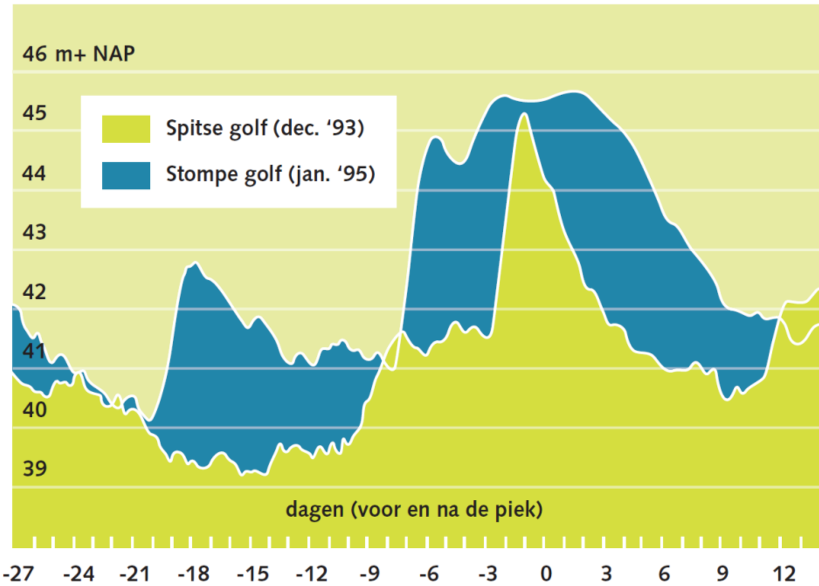


Figure 11 - Water levels at Borgharen the days before and after the flood waves of 1993 and 1995 (source: Rijkswaterstaat)

In Table 5 is represented that the time a flood wave needs to move from Borgharen to Maasbracht (beginning of canal section Linne) depends on the discharge. The time a flood wave needs to reach Maasbracht is between 9.4 and 21.2 hours. It is remarkable that the duration a flood wave to pass the section between Maasbracht and Roermond (= canal sections Linne and Roermond) is relative large. In this section are the Maasplassen located, it can be seen that the Maasplassen have a certain damping effect on the discharge of high water waves.

The Grensmaas project has upgraded the Border Meuse, the bottom is deepened and the ecological situation improved. The works on the Border Meuse should be finished in 2022. The effect of the Grensmaas project and increasing future discharges have in all probability effect on the duration of flood waves to reach Maasbracht. It should be investigated if the duration between 9.4 to 21.2 hours is still reliable. For the progress of this report is the duration of a flood wave to reach Maasbracht assumed at 8 hours. It has to be noticed that this assumption should be investigated to determine the accurate duration time of the flood wave to reach Maasbracht.

Table 5 - Duration of flood waves on the Meuse for discharge Q (source: Rijkswaterstaat)

Q [m^3/s]	Duration of the flood wave relative from Borgharen [hour]			
	Maasbracht (46 km from Borgharen)	Roermond (65 km from Borgharen)	Grave (160 km from Borgharen)	Lith (184 km from Borgharen)
600	9.4	16.1	39.3	48.2
700	9.4	16.0	38.2	46.8
800	9.4	16.1	37.5	46.8
900	9.4	16.5	38.2	46.8
1000	9.8	17.5	39.7	48.6
1200	10.7	19.8	44.0	54.5
1400	12.0	22.5	50.1	62.8
1600	13.7	26.3	60.9	71.4
1800	15.0	29.0	69.2	79.8
2000	16.6	31.5	75.5	85.4
2500	19.2	34.9	83.8	92.1
3000	20.8	36.1	84.0	91.8
3500	21.2	35.2	80.7	87.5
3800	21.2	34.8	78.9	85.3
4000	21.2	34.5	77.1	83.9

3.1.2 Discharging objects

This section treats constructions that are used to discharge water from canal section Linne to canal section Roermond. The objects are:

- The weir (section 3.1.2.1)
- The hydroelectric power plant (section 3.1.2.2)
- The fish ladder (section 3.1.2.3)
- The spillway (section 3.1.2.4)

3.1.2.1 Weir

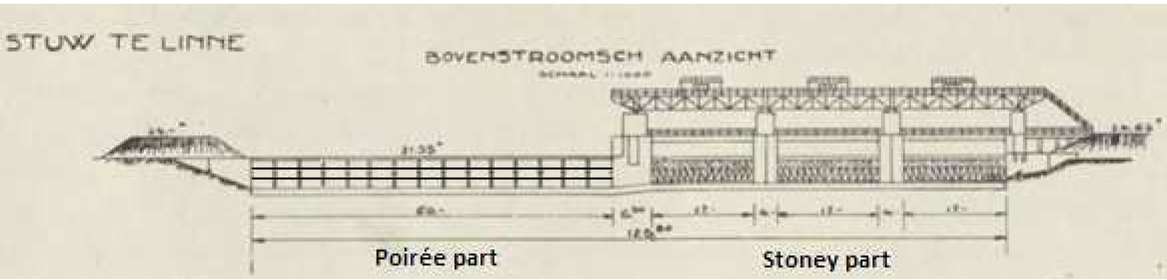


Figure 12 - Layout of weir Linne (source: Rijkswaterstaat)

The weir of Linne is located in the Meuse nearby the small village Linne. This weir is the second weir downstream of the Dutch - Belgian border. The function of the weir is to dam a part of the Meuse for navigational purposes, the function differ with the discharge of the Meuse. The distinction is made between low/average discharges and high discharges:

- *Low and average discharge*
  During low discharges the weir is maintaining the water level in the canal section Linne. The average water level in the canal section is 20.80 m +NAP. Maintaining the water level is important for navigation during low and average discharges. The surplus of water entering the canal section is discharged to canal section Roermond by the Stoney part of the weir and hydroelectric station. The Poirée part is in this situation totally closed. Up to 500 m^3/s the

water surplus in canal section Linne is discharged by the hydroelectric station and the Stoney part of the weir.

- *High discharge*
  The weir has no function during high discharges. During high discharges is water discharges to canal section Roermond by the weir. The gates are totally removed for discharges of 1200 m^3/s. For higher discharges the area around the weir will flood and the water will also flow around the weir. It takes according to Table 5 between 9.4 and 21.2 hours before a measured high water wave at Maastricht St. Pieter reach canal section Linne (Rijkswaterstaat, Hoogwater op Rijn en Maas, 2007).

3.1.2.1.1 Poirée part

The weir is a combined Poirée-Stoney system as represented in Figure 12. In this section is the Poirée part discussed. The Poirée weir is a variant on the needle weir. The weir is developed by the French engineer Charles Poirée. The Poirée weir was a large improvement in weir construction because this type of weir is able to accommodate on changing conditions. The disadvantage of this weir type is that the regulation is inaccurate. The Poirée weir consists of yokes which are connected by hinges to the bottom of the weir. Three rows (upon each other) of 15 partitions are placed against the yokes, the partitions together form a wall perpendicular to the river cross section. The wall pushes up the water. The first step in lowering of the weir is the removal of the partitions by means of a crane. The partitions are lifted from the jokes and stored in the storage beside the weir construction. The partitions are removed one by one and row by row. After the partitions are removed the removable plateau parts on top of the yokes will be removed and the yokes will be lowered to the bottom of the weir (Berger & Mugie, 1994). An exact duration for removing the Poirée parts is hard to determine because a flood wave at the Dutch - Belgian border (at measuring station Maastricht St. Pieter) is preceded by an increase in discharge as can be seen in Figure 11. As result of the increasing discharge will be started with removing some partitions of the Poirée weir to maintain the water level of the canal section.

A representation of a Poirée part is represented in Figure 13, it has to be noticed that the figure is a not a representation of the Poirée part of weir Linne (weir Linne contains three rows of partitions instead of two)

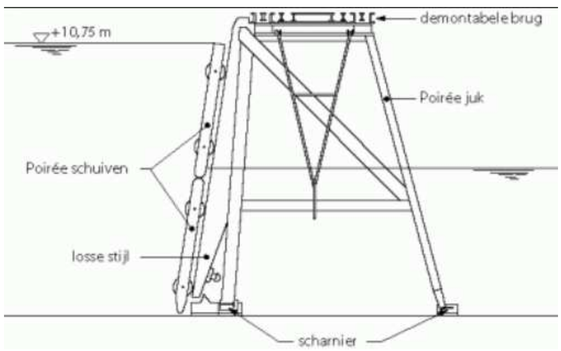


Figure 13 - Cross section of a Poirée weir (souce: CoFlex bouw en infra)

The removal procedure of the first two of the three partition rows is presented in Figure 14 and Figure 15 (van Mazijk, Onderzoek betreffende afvoerberekeningen gebaseerd op stuwgegevens van stuwen Belfeld en Sambeek, 1977). The left side of the Poirée part presented in Figure 14 and Figure 15 is the side of the Stoney part, at the right side the weir bank is located. It can be seen in the figures that the first removed partitions are located on the bank side of the Poirée part. The bank side partitions are first removed to obtain a more evenly distributed discharge over the total weir construction (Poirée and Stoney).

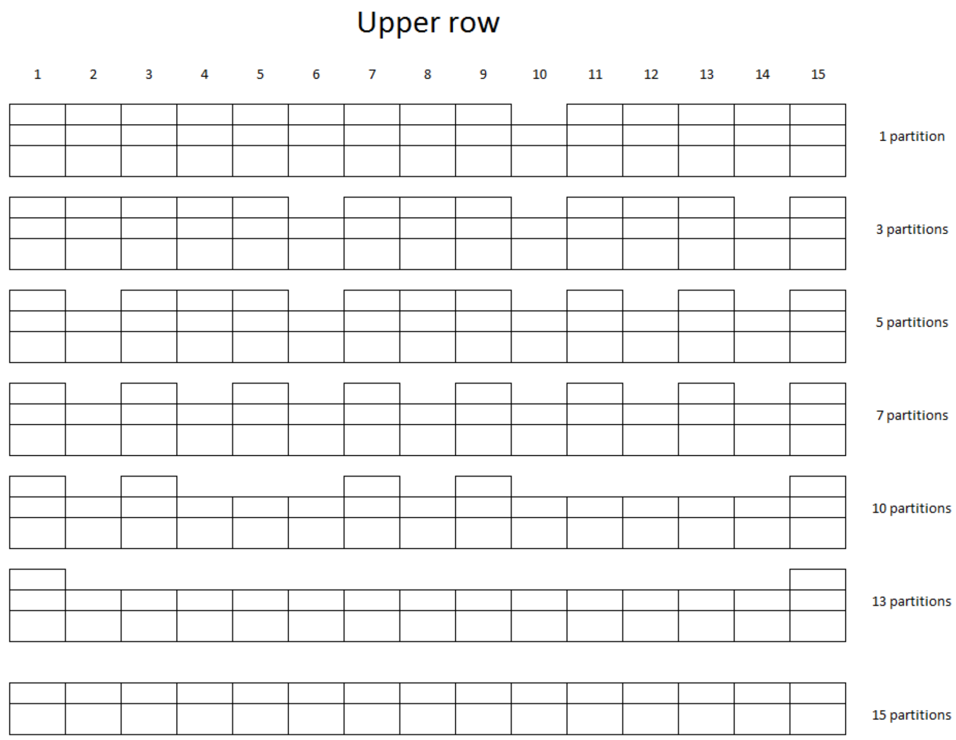


Figure 14 - Removal of flaps of the upper row

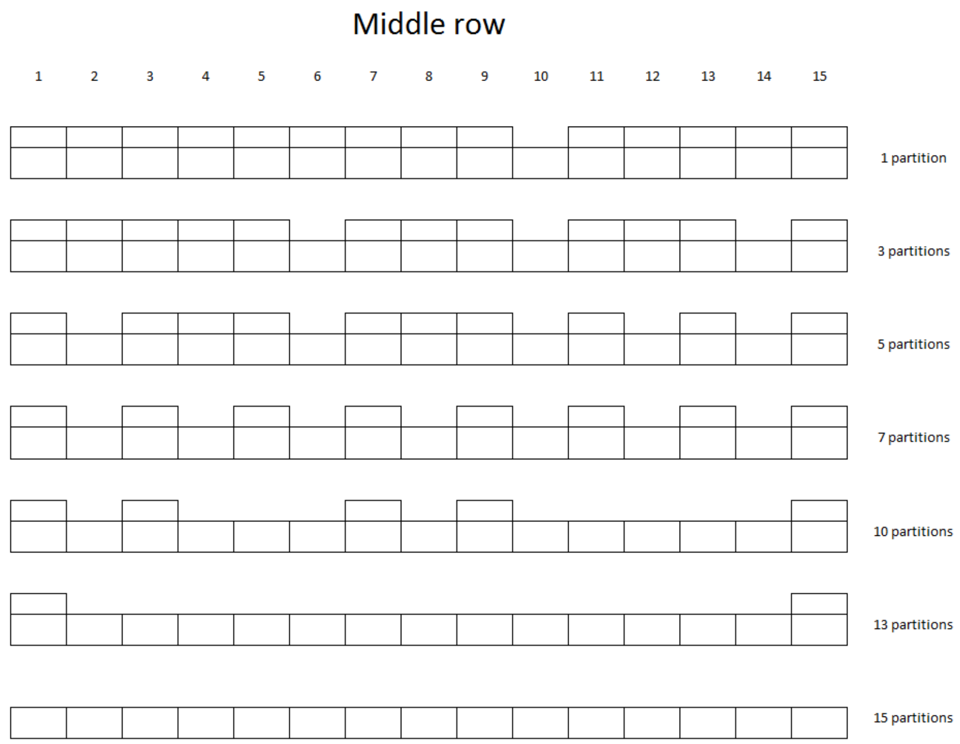


Figure 15 - Removal of flaps of the middle row

Every removed partition result in a slight change of the discharge coefficient m as presented in Equation 1. The resulting discharge coefficients corresponding to the removal of Poirée partitions as according to Figure 14 and Figure 15 is presented in Table 6 (Rijkswaterstaat, IJking stuwen te Linne, Roermond, Belfeld en Sambeek, 1979).

Table 6 - Discharge coefficients of the Poirée part (source: Rijkswaterstaat)

Removed partitions	Discharge coefficient m	
	Upper row	Middle row
1	0.623	0.645
3	0.623	0.645
5	0.624	0.646
7	0.625	0.649
10	0.628	0.652
13	0.634	0.667
15	0.640	0.685

Figure 16 represents the discharge of the Poirée part for the removal of the first two rows as presented in Figure 14 and Figure 15. The figure is constructed by means of Equation 1 and Table 6. The upstream water level is taken at h_{up} 4.85 m (difference between the upstream water level (=water level of canal section Linne) and the weir bottom) and the downstream water level as h_{down} = 0.90 m (difference between the downstream water level (=water level of canal section Roermond) and the weir bottom). The width B in Equation 1 presents the discharge width of the weir. The width of 1 partition is 4 m (=60 m /15 partitions = 4 m per partition), so if two partitions are removed the width becomes 8 m etc. This is a simplified model of the discharge over the Poirée part. The simplification is reflected in to aspects:

- in the real situation the up and downstream water level will change during increasing discharges
- Contraction of the flow lines occur at the location of the openings in the Poirée part. Due to the contraction the effective width becomes smaller than the actual discharge width.

$$Q = m * \frac{2}{3} * \left(h_{up} - h_{cr}\right) * B * \sqrt{\frac{2}{3} * g * \left(h_{up} - h_{cr}\right)}$$

Equation 1 - Discharge for an overflow weir

The high peak in the middle of Figure 16 represents the removal of the first partition of the middle row. This is the result of the increased difference between the top of the second row of and the upstream water level. According to Equation 1 the discharge increases for a larger difference between crest and water level. The course of the discharge represented in Figure 16 is relatively rough. It can clearly be seen that the Poirée part creates a coarse discharge development when the weir is opened. It can also be seen that the development of the discharge is not constant. The figure is obtained by measurements of weir Linne (Rijkswaterstaat, IJking stuwen te Linne, Roermond, Belfeld en Sambeek, 1979)

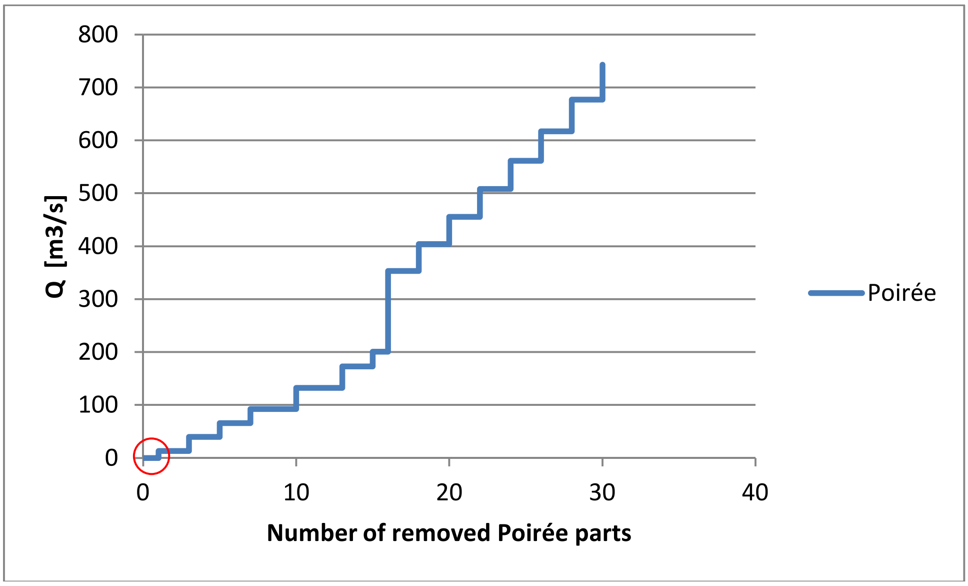


Figure 16 - Discharge of the Poirée part for the removal of partitions

3.1.2.1.2 Stoney part

The Stoney weir is developed by the British engineer Francis Stoney. The Stoney weir consist of mechanically operating gates which are placed between two pillars. The gate consists of two slides, a higher and a lower slide, the slides are able to operate separately. Separate operation of the slides contains the opportunity to obtain a free surface flow or a submerged flow. The advantage of this type of weir in comparison with the Poirée weir is the possibility of accurate discharge regulation (Berger & Mugie, 1994). The Stoney part consists of three openings of 17 m and is the fine regulating part of the weir (Berger & Mugie, 1994). The fine regulation by the Stoney part operates as follows:

- The Stoney part regulates the water level of canal section Linne by means of lifting and lowering of the gates
- When the discharge capacity of the Stoney gates is reached, is discharge regulation not possible anymore. Partitions of the Poirée part are removed to increase the discharge capacity of the weir. The accurate discharge regulation is again managed until the discharge capacity of the Stoney gates is reached. This procedure is repeated until the total Poirée part is removed.

For low and average discharges a water level of NAP + 20.80 m is maintained with tolerances between NAP + 20.70 m and NAP + 20.90 m (van der Veen, 1989). The weir gates are entirely removed for discharges higher than 1200 m^3/s (Kranenbarg & Kemper, 2006). An impression of a Stoney weir is represented in Figure 17.

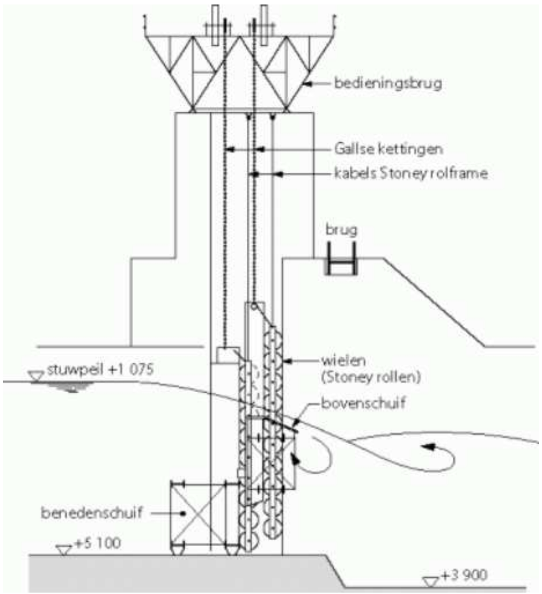


Figure 17 - Cross section of a Stoney weir (source: Coflex bouw en infra)

The discharge regulation of the Poirée weir has a stepped course. Every time when a partition is removed a the discharges increases by a certain amount of water so a precise control of the water level is not possible for a Poirée weir. To obtain a more precise control of discharge are the Stoney gates used. The Stoney gates are lowered in steps of 10 cm (van Mazijk & van Knippenberg, IJking stuwen te Linne, Roermond, Belfeld en Sambeek, 1978). The discharge accuracy of the Stoney gates is 5% (van Mazijk, Onderzoek betreffende afvoerberekeningen gebaseerd op stuwgegevens van stuwen Belfeld en Sambeek, 1977). A simplification of the effect of the Stoney weir is represented in Figure 18. The blue line in the figure represents the removal of 1 Poirée as is accentuated with the red circle in Figure 16. The red and green lines represent the discharge accuracy of the gates of the Stoney weirs. The accuracy of the Stoney gates with respect to the Poirée weir is clearly visible, the Stoney gates create a more smooth discharge transition for removed Poirée parts.

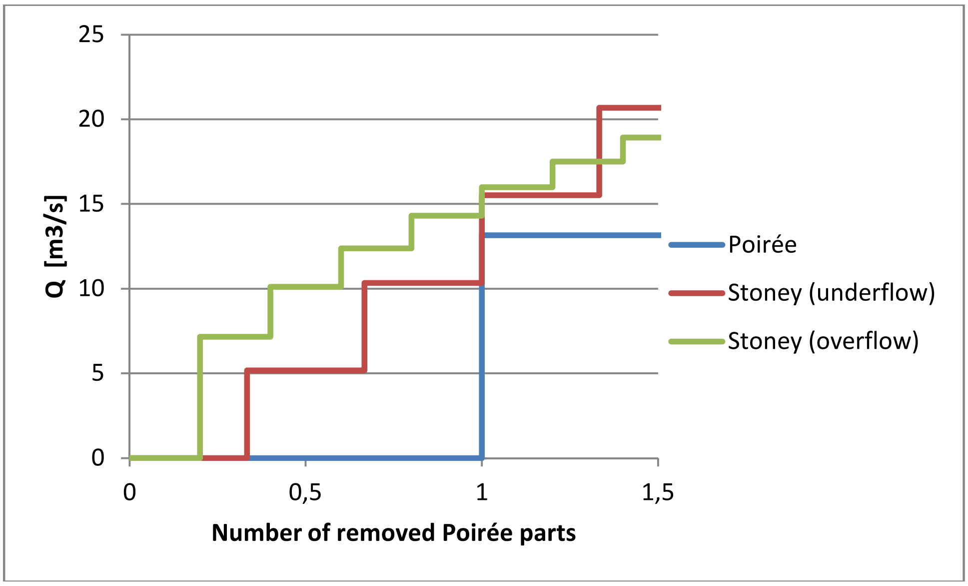


Figure 18 - Discharges control of Poirée and Stoney gates

3.1.2.2 Hydroelectric power plant

On the south side of the weir complex the hydroelectric power plant is located. The plant contains 4 turbines, every turbine generates a maximum of 2.87 MW. The maximum head difference of the power plant is 4 m. During low and average discharges water should be discharged by the hydroelectric power plant instead of the weir. The plant is able to discharge up to a maximum of 500 m^3/s. At discharges between 500 and 800 m^3/s is 500 m^3/s discharged by the plant and the rest by the Stoney part of the weir. The plant is closed when discharges become higher than 800 m^3/s because the head difference between upstream and downstream becomes too small. The discharge is 287 days a year lower than 500 m^3/s, 45 days between 500 - 800 m^3/s and about 33 days a year higher than 800 m^3/s. The power plant generates 11.50 MW, more than 10000 households are served with sustainable energy (Bruijs, 2004).

3.1.2.3 Fish ladder

The fish ladder is located on the south side of the weir complex and offers the opportunity for fish to pass the weir complex. The attracting stream of the fish ladder directs fish to the ladder. The ladder has a length of 215 m and a height difference of 4.05 m. The ladder is divided in 17 steps, the steps have been made of natural materials like rocks and gravel. The mean discharge of the fish ladder is about 2 m^3/s during average discharges, during extreme discharges will the fish ladder be totally submerged (Berger & Mugie, 1994).

3.1.2.4 Spillway

The spillway of Linne is located on the north side of the weir. The function of the spillway is to transport water around the weir during high discharges. The crest of the spillway is located at 21.30 m +NAP. The height of the crests corresponds with Meuse discharges between 1450 to 1550 m^3/s at measure point Maastricht St. Pieter (Kurstjens, Peters, & Calle, 2008). The river loop behind the weir and spillway is called the Lus van Linne. The area functions as floodplain during high discharges so that a higher discharge profile is created. On top of the spillway is a bicycle path created, during discharges higher than 1550 m^3/s is the bicycle path out of service.

3.1.3 Navigation

The canal section of Linne is part of the main navigational route on the Meuse. The required vessel class in the canal section Linne is therefore class Vb. It is important that navigation is maintained as long as possible during extreme discharges. Navigation is impossible during extreme water levels and extreme high discharges. When the water drops below 20.65 m +NAP is no sufficient navigational draught present in the canal section (Rijkswaterstaat, Hoogwater op Rijn en Maas, 2007). Navigation is still possible however for not totally loaded vessels or vessels smaller than class Vb. At discharges higher than 2500 m^3/s no navigation is allowed on the Meuse.

3.1.3.1 Harbors

The canal section Linne contains two types of harbors, commercial harbors and recreational harbors. The canal section contains 8 recreational harbors and two commercial harbors. The recreational harbors are not discussed in this report. The commercial harbors of the canal section are located in Maasbracht and Wessem. In 2007 the municipalities of Maasbracht, Thorn and Heel were brought together in a new municipality called Maasgouw. The harbors of Maasbracht and Wessem are since that moment property of the municipality of Maasgouw. The combined harbor is one of the largest inland harbors in the province of Limburg and the largest harbor for sand and gravel transshipment in Limburg. The annual transshipment of the harbor is 4.597.026 tons per year (Provincie Limburg, Netwerkanlyse vaarwegen en binnenhavens, 2008).

3.1.4 Locks

The canal section of Linne contains four locks. The locks provide vessels access between two areas with different water levels. The locks connected to canal section Linne are:

- *Lock Maasbracht*
  Lock Maasbracht separates the Julianakaal and the Meuse. The average water level difference is 11.85 m between the Julianakanaal and the Meuse. Due to the large water difference the height difference of lock Maasbracht is the largest in the Netherlands. The lock

complex consists of three chambers, the largest chamber is the eastern chamber. The chamber is suitable for class Vb vessels. Lock Maasbracht is a main lock of the navigational route. During normal conditions (sufficient discharge) 13 m^3/s is leaked to canal section Linne. The lock complex contains a pumping station, during extreme low discharges it is impossible to feed the Julianakanaal on the upstream side. Water from canal section Linne is pumped in the Julianakanaal to maintain the water level. During extreme low discharges 7 m^3/s is leaked to canal section Linne, 9 m^3/s pumped into the Juliankanaal to maintain the water level in the Julianakanaal. This is necessary because during extreme low discharges no water at Borgharen is directed to the Julianakanaal. (Berger & Mugie, 1994).

- *Lock Panheel*
  Lock Panheel is the transition between the Meuse and the Kanaal Wessem-Nederweert. The lock provides entrance to the Kempische Kanalen, the canals inside 'de Grote Ruit'. This region is call the Kempen. The water level difference between the Meuse and Kanaal Wessem-Nederweert is 7.85 m (Rijkswaterstaat, Kanaal Wessem-Nederweert, 2014). Lock Panheel is suitable for vessels of the CEMT II class (a vessels of this class is called a Kempenaar). The leakage of lock Panheel is due to reservoirs reduced to 1 m^3/s. The lock complex contains a pumping station. The station is used during periods of low discharges. Volumes of 4 to 6 m^3/s are pumped in Kanaal Wessem Nederweerd to compensate leakages of lock Panheel and irrigational purposes along the Kempische Kanalen (Berger & Mugie, 1994).
- *Lock Heel*
  Lock Heel is located in the Lateraalkanaal. Lock Heel is the transition between canal section Linne and canal section Belfeld by means of the Lateraalkanaal. The average water level difference between the canal section Linne and the Lateraalkanaals is 6.7 m. The lock is frequently used by navigation due to two reasons:
    - The Lateraalkanaal provides a more quick passage than the route trough canal section Roermond.
    - The Lateraalkanaal is compared to canal section Roermond suitable for vessel class Vb.
- *Lock Linne*
  Lock Linne is the transition between canal section Linne and canal section Roermond. The lock is mainly used by recreational boating instead of commercial transport as is presented in Appendix B. The average water difference between the two canal sections is 3.95 m.

3.1.5 Remaining objects in canal section Linne

3.1.5.1 Clauscentrale

The Clauscentrale is the largest energy power plant in the Netherlands. The plant generates energy out of gas. The plant is located nearby Maasbracht. The plant generates 1945 MW and serves more than 2 million households of energy. The power plant is using water from the Meuse to cool the installation. Since the 1 July 2014 is the Clauscentrale closed because the power plant is not profitable to the current energy market. The power plant is reopened when the environment is more economically viable (Essent, 2014).

3.1.5.2 Maasplassen

The Maasplassen is a 3000 ha large continuous recreational and water sports area. The area is located in central Limburg between villages and cities like Heel, Maasbracht and Roermond. The Maasplassen were created in the 20th century by excavation of gravel. Due to the excavations, lakes were created along the Meuse. Surrounding municipalities use the lakes to create recreational facilities by constructing marinas, campsites and small beaches.

The Maasplassens are partly located in the canal sections Linne, Roermond and Belfeld. Part of the lakes are directly connected with the Meuse, the other part has no open connection with the Meuse. The open connected lakes result in a damping effect of the water level change due to varying discharges. The damping effect is the result of the increase of water storage of the area.

3.1.6 Discharge management

At discharges lower than 500 m^3/s no water is flowing over the weir but is discharged by the hydropower plant. By discharges higher than 500 m^3/s the weir is activated to discharge the water together with the hydropower plant and the fish ladder. For higher discharges than 500 m^3/s the procedure of the combined Stoney and Poirée part (as described in section 3.1.2.1.2) will be used. At discharge higher than 800 m^3/s the hydropower plant is closed because the head difference between up and downstream becomes too low. At discharges higher than 1200 m^3/s all the partitions of the Poirée part are removed and the yokes of the Poirée part lowered. Also the Stoney gates are lifted out of the water and water is discharged over the Poirée and Stoney part. When discharges become larger than 1550 m^3/s also the spillway discharges water. A summary of the discharge management is given in Table 7.

Table 7 - Weir management of weir Linne

Discharge [m^3/s]	Discharge construction
< 500	Hydropower plant
500 - 800	Hydropower plant + Stoney weir + Poirée weir (Poirée + Stoney weir partly removed)
800 - 1200	Stoney weir + Poirée weir (Poirée + Stoney weir partly removed)
1200 -1550	Stoney weir + Poirée weir (Poirée + Stoney weir both lifted out of the water)
> 1550	Stoney weir + Poirée weir (Poirée + Stoney weir both lifted out of the water) + Spillway

3.2 Functions of canal section Linne

The system of canal section Linne has several functions. The functions are divided into main functions and secondary functions. The main functions are the most important tasks of the system to perform in a right way. The system of the Meuse has two main functions, the main functions are:

- *Transport of water*
  Water has to be transported from the upstream boundary (Border Meuse up to Maaseik) to the downstream boundary (weir Linne). The system should be able to transport a certain amount of water in a safe manner according to safety norms.
- *Navigation*
  The system should be able to provide navigational transport of the Vb class from the upstream to the downstream boundary. To enable navigation the waterway should have sufficient draught width and vertical clearance.

The secondary functions of the canal section are:

- *Feeding of canals*
  During extreme low discharges of the Meuse water from the canal section is pumped into the Julianakanaal.
- *Generation of energy*
  The hydroelectric power plant of Linne requires a discharge of water to generate energy out hydropower
- *Supply water for industry*

The Clauscentale is the only industry in the canal section which uses water from the Meuse. The power station is not operable at the moment but is taken into account because in the future the power station will probably be used.

- Functions behind scoop of this report are:
  - *Agriculture*
  - *Recreation*
  - *Fishing*
  - *Irrigation water*
  - *Environment*
  - *Living*
  - *Ecological purposes*
  - *Landscape*

3.3 Boundaries of the system of canal section Linne

3.3.1 System boundaries

The system boundaries define which parts are included in the system of canal section Linne and which are behind scope of the system. The system boundary conditions are:

- Border Meuse up to Maaseik (upstream boundary)
- River banks
- River bottom
- Lock Panheel
- Lock Heel
- Lock Linne
- Lock Maasbracht
- Weir Linne (downstream boundary)
- Hydroelectric station Linne (downstream boundary)
- Fish ladder (downstream boundary)

3.3.2 Boundary conditions

- Discharge of the Border Meuse
- Water depth in the canal section
- Discharge capacity of the hydroelectric power plant
- Discharge capacity of weir Linne
- Leakage of lock Linne
- Leakage of lock Heel
- Leakage of lock Panheel
- Leakage of lock Maasbracht

3.4 Requirements for canal section Linne

The requirements of the design step are presented in Appendix E. Only the main and most important requirements resulting from the list are presented in this section. The main requirements have the largest impact for the design solution. The requirements are:

- For discharges higher than 1200 m^3/s should the weir gates be totally removed.
- Discharges below 500 m^3/s have to be discharged by the hydroelectric power plant
- 4 to 6 m^3/s Should be pumped into the Kanaal Wessem-Nederweert during low discharges of the Meuse.
- During extreme low discharges must 9 m^3/s be pumped in the Julianakanaal
- During water levels below 20.65 m +NAP is no navigation not possible for fully loaded vessels in the canal section Linne.

3.5 Behavior analysis

The behavior analysis investigates the effect of the Maasplassen to the change of water levels in the canal section. The behavior of the water levels are investigated by means of a mass balance. Water level changes during high discharge have influence on the time that is necessary to opening and closure of the weir. Water level changes during low discharges have effect on navigation, the accuracy of the weir discharge is important in this situation. The lakes directly connected with the Meuse have a damping effect on the velocity of the water level changes. The lakes taken into account are represented in Table 8.

Table 8 - Maasplassen in canal section Linne

Lakes connected to the Meuse	Surface area [km^2]	Lakes unconnected to the Meuse	Surface area [km^2]
Herenlaak	0,823	Dilkensplas	0,160
Schroevendaalseplas	0,328	Teggerse Plas	0,052
Maasplas Kinrooi (groot)	2,746	Boschmolenplas	1,011
Maasplas Kinrooi (klein)	0,539	De Lange Vlieter	1,240
Huyskensplas/De Kist	0,404		
Maasplas Stevensweert	1,582		
De Grote Hegge	1,582		
Molengreend	0,354		
De Slaag	0,214		
Polderveld	0,464		
Tesken	0,174		
Sint-Antoniusplas	0,108		

Damping is the result of the storage capacity of the lakes. According to the mass balance in Equation 2 the effect can be explained.

$$\Delta Q = O\frac{dh}{dt}$$

Equation 2 - Mass balance

$\frac{dh}{dt}$ = velocity of the water level change [m/s]

For a constant discharge, a large storage capacity results in a slow change of the water level and a small storage capacity for a quick rise of the water level. It is expected according to the KNMI'14 climate scenarios that the high discharges will increase and the low discharges will decrease. The behavior of the water level in relation to the increasing high and low discharges is investigated. The effect of the storage capacity of the connected lakes is investigated to obtain a better understanding of the effect of water level changes in the canal section.

The model that is used to analyze the behavior is of the water level is the mass balance. The mass balance is used to analyze the water level behavior during extreme high and extreme low discharges of the Meuse. The used discharges are the discharges according to the KNMI'14 climate scenarios. More details about the scenarios are represented in Appendix D. The behavior of the extreme high and extreme low discharge are analyzed for three different variants. The variants should give a clear view about the behavior of the Maasplassen with respect to water level changes in the canal section. The variants are represented in Figure 19. The variants are:

- *Variant 1: Maintain the current situation*

In this situation the current storage of the canal section is maintained, this means that the Maasplassen which are connected to the canal section remain connected.

- *Variant 2:* *Disconnect the lakes in the canal section from Meuse*
  In this variant all Maasplassen which are connected with the canal section will be disconnected with the canal section. This variant is taken into account in order to obtain insight in the effect of the Maasplassen.
- *Variant 3:* *Connect all lakes in the canal section with the Meuse* (also the unconnected lakes of the current situation)
  In this variant all Maasplassen that in the current situation are not connected with the canal section will also be connected. This results in a increase of the storage of the canal section.

It has to be noticed that the results of the behavior analysis do not represent the real situation. River banks and retention areas are not taken into account. The velocities of the water level changes in the real situation will be smaller. However the banks and retention areas are omitted in all three variants. However relative to each other the variants represents a clear view about the effect of the Maasplassen on the water level changes.

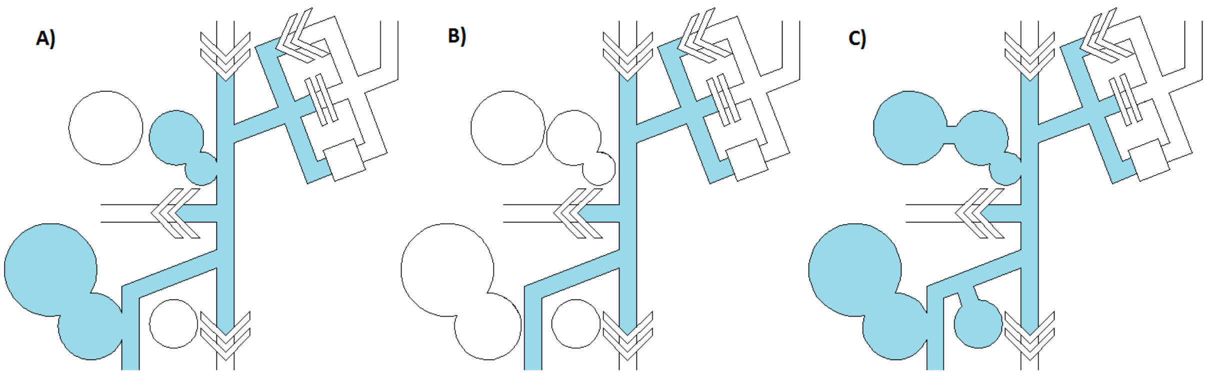


Figure 19 - Variants for investigation of the effect of the Maasplassen; A) Variant1 ; B) Variant 2; C) Variant 3

The behavior analysis is done for high and for low discharges. As was mentioned in section 3.1.1.1 is the assumed duration before the high water peak reached weir Linne is 8 hours. Before the high water peak reaches the canal section the weir should be opened. The analysis of the high discharge investigates whether the time to open the weir could be increased due to the presence of the Maasplassen. The accuracy of the discharge over the weir is of minor importance. The important aspect is that high discharge can pass the weir without disturbances. The time of opening could have effect on the type of the future weir.

The analysis for the low discharges is focusing on the accuracy of the weir. It is investigated if during low discharges the discharge accuracy of the weir is sufficient. The inflow and outflow due to leakage of locks and weirs are taken into account. The effect of the Maasplassen on water level in the canal section with respect to the accuracy of the weir is investigated. A sufficient water level in the canal section is required for navigation.

3.5.1 High discharges

The prediction of high waters is important for the weir management because the weir should be opened before the high water peak reaches the weir. The accuracy of the discharge over the weir is of minor importance. The important aspect is that high discharge can pass the weir without disturbances. High discharges can be predicted 6 to 12 hours before the high water wave reach the Dutch border (Rijkswaterstaat, Hoogwater op Rijn en Maas, 2007). In the current situation the opening operation of the weir is started if discharges become higher than 500 m^3/s measured at measuring point Maastricht St. Pieter. Canal section Linne is a relative small canal section so contains a relatively small storage capacity. The small storage capacity results in relative quick water level changes during high discharges. High discharges are evaluated until the water level in the canal section is 21.30 m +NAP. Above this water levels the river banks and the spillway are activated, the storage of the river banks is equal in all three variants. In the model discharges between 3000 and 4600 m^3/s are taken into account, the discharges according to the KNMI '14 scenarios. More information about the scenarios can be found in Appendix D.

3.5.1.1 Sub variants for high discharges

Two weir configurations are taken into account, namely opened and closed configuration. The operations are applied to the variants. The sub variants are taken into account to investigate the effect of the Maasplassen on the velocity of water level rise. A slower velocity of the water level rise results in a longer time until a water level of 21.30 m+ NAP is reached. A slower rise of the water level results in more time to remove the gates from the water.

- *Sub variant A: Closed configuration*
  Closed configuration determines the water level rise during high discharges for a complete closed weir. The closed configuration corresponds with a not responding weir. When the high discharge reaches the canal section and the weir is closed will the water level in the canal section rise quickly. This is the most unbeneficial scenario during high discharges. The configuration is represented in Figure 20 A.
- *Sub variant B: opened configuration*
  Opened configuration determines the water level rise in the canal section for a completely opened weir. The open configuration corresponds with and perfect responding weir (weir could be opened before the high water peak reaches Linne). The totally opened configuration results in the water level changes in the canal section. This configuration is the most beneficial during high discharges. A high discharges needs according to the assumption of section 3.1.1.1. approximately 8 hours (after passing the Dutch - Belgian border) to reach canal section Linne. The duration of flood waves and the corresponding discharges are represented in Table 5. The current weir should be opened between 0 and 8 hours to obtain the desired condition of opened configuration. The configuration is represented in Figure 20 B.

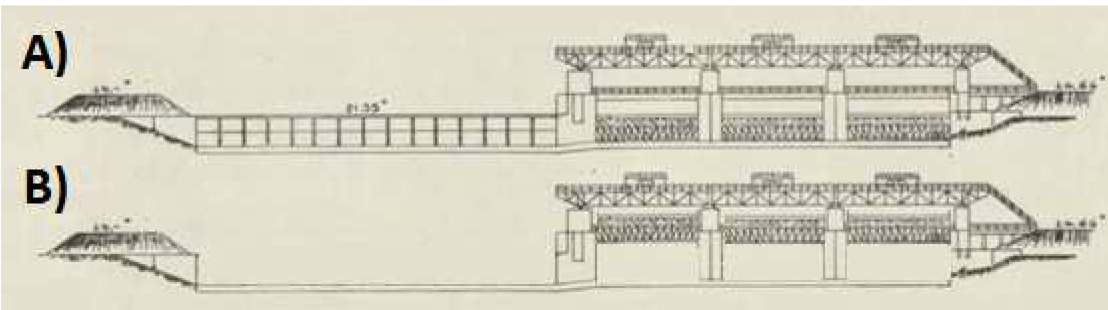


Figure 20 - Weir configurations A) Closed configuration; B) Opened configuration

The duration until the water level reaches 21.30 m +NAP is calculated for both configurations. Also the differences in time between both configurations is investigated. The time differences give insight if the damping effect is substantial and the effect of the damping on high discharges. The evaluation of the three variants for high discharges are represented in Appendix G. In this section the results of the behavior of the water levels for the three variants are summarized to obtain insight in the effect of the damping effect of the Maasplassen in the canal section. The results are summarized as follow:

- The velocity of the water level rise in the third variant is the most beneficial because the velocity of the water level rise is lower than in the other variants. The third variant contains the largest storage capacity, this corresponds with Equation 2.
- The velocity of the water level rise in the first variant is a little higher than in the third variant, however the storage capacity of the third variant is not substantial larger than the capacity of the first variant. This results in a velocity comparable to the velocity of the third variant.
- The water level rises the fastest in the second variant, the reason is the low storage capacity of the variant.
- For increasing discharges the difference between a perfect responding and a not responding weir becomes smaller. The damping effect of the lakes has less influence on high discharges because canal section Linne is relative small.

Based on the summarized results the first variant is most beneficial for the high discharges, the current situation is maintained. This variant is most beneficial for the high discharges because increasing the storage area of the canal section has less influence on the increase of the water level rise. The graphs corresponding to the first variant are represented in Figure 21. The time at the horizontal axis in Figure 21 begins at 8 hours. The reason is that a flood wave needs 8 hours to travel from measuring point Maastricht St. Pieter to Maasbracht (beginning of canal section Linne) according to section 3.1.1. The figure represents that the water level rise is faster for the closed than for the opened condition. The time difference of the water level rise of the two variants is about 45 minutes until a water level of 21.3 m +NAP (water level for which the spillway is activated) is reached. For higher water levels the discharge is not anymore controlled by the weir. The water level will rise relatively quick when a flood wave arrives at the canal section, the lakes have a certain damping effect on the water level rises. When no lakes are connected the water level will in shorter than 15 minutes reach 21.3 m +NAP (opened condition). The lakes increase the rising time of the water level 3 times compared to the situation without lakes.

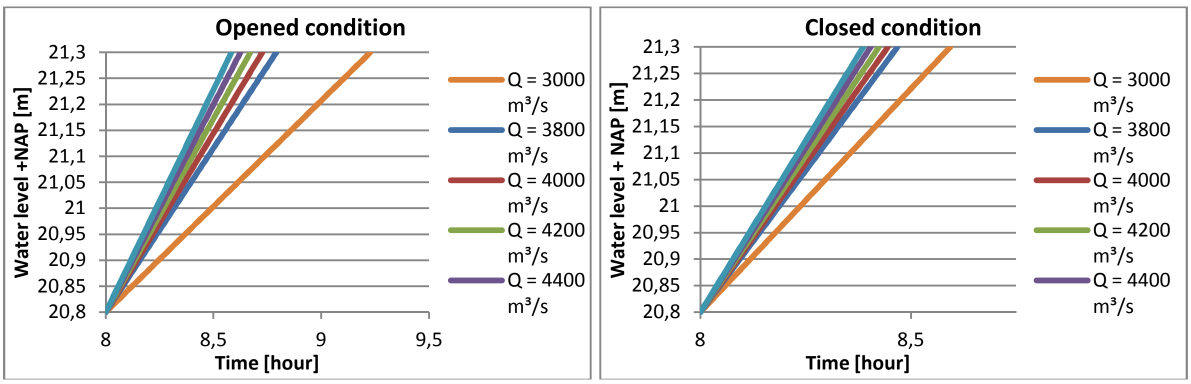


Figure 21 - Water level rises for variant 1 during high discharges; Left: water level rise for the opened condition (sub variant a for high discharges); Right: water level rise for the closed condition (sub variant b for high discharges)

3.5.2 Low discharges

There is spoken about low discharges if the discharge of the Meuse in the Netherlands becomes lower than 30 m^3/s at measuring point Maastricht St. Pieter. This occurs mainly between the months July and September. When discharges at this measuring point become smaller than 36 m^3/s a water shortage in the canal section arises (Breukel, Silva, van Vuuren, Botterweg, & Venema, De Maas - Verleden, heden en toekomst, 1992). The discharge regulation during low discharges is managed by the hydroelectric station and the Stoney part of the weir. When discharges are extremely low, the discharge is only regulated by the Stoney part of the weir because small discharges are not able to drive the turbines of the hydroelectric station.

In section 3.1.1 was mentioned that a minimum discharge of 10 m^3/s over the Border Meuse is required to maintain the ecological situation of the Border Meuse. The 10 m^3/s that enters canal section Linne should be discharged by the weir to canal section Roermond. The minimum discharge capacity of the weir should therefore be 10 m^3/s. The Stoney gates can be lifted in steps of about 10 cm (Rijkswaterstaat, IJking stuwen te Linne, Roermond, Belfeld en Sambeek, 1979). As can be seen in Figure 18 the discharges of the overflow discharge is more sophisticated that of the underflow discharge. The accuracy of the overflow gate is calculated according to Equation 1. The discharge coefficient of the Stoney gates is determined according to Equation 4 (van Mazijk, Onderzoek betreffende afvoerberekeningen gebaseerd op stuwgegevens van stuwen Belfeld en Sambeek, 1977). The discharge coefficient becomes for a lowering of the upper gate with 10 cm ($h_{cr,Stoney}$ = 20.70 m +NAP) m = 0.95.

$$m = \left(1 - \frac{1}{(h_{up} - h_{cr,Stoney} + 1)} + \frac{4(h_{up} - h_{cr,Stoney})}{250}\right) + \left(0.7 + 0.05(h_{up} - h_{cr,Stoney})\right)$$

Equation 3 - Discharge coefficient of the Stoney gates

The discharge of the Stony gates are presented by Equation 4. The equation is valid for the total Stoney part. The first part of the equation presents the effective discharge width. The effective discharge width is the result of the total discharge width including the contraction of the water flow.

$$Q = \left(51 - 0.2(h_{up} - h_{cr,Stoney})\right) \frac{2}{3}\, m \sqrt{2\, g\, (h_{up} - h_{cr,Stoney})^3}$$

Equation 4 - Discharge of the Stoney gates

The resulting discharge accuracy of the Stoney gates becomes Q = 6 m^3/s. The discharge accuracy is smaller than the minimum required discharge over the Border Meuse (10 m^3/s), the accuracy is sufficient. In the actual situation not all the water is discharged by the weir. Due to leakage of the weir, locks and the fish ladder water enters and leaves the canal section. The inflow and outflow of the canal section is presented in Table 9 (Breukel, Silva, van Vuuren, Botterweg, & Venema, De Maas - Verleden, heden en toekomst, 1992).

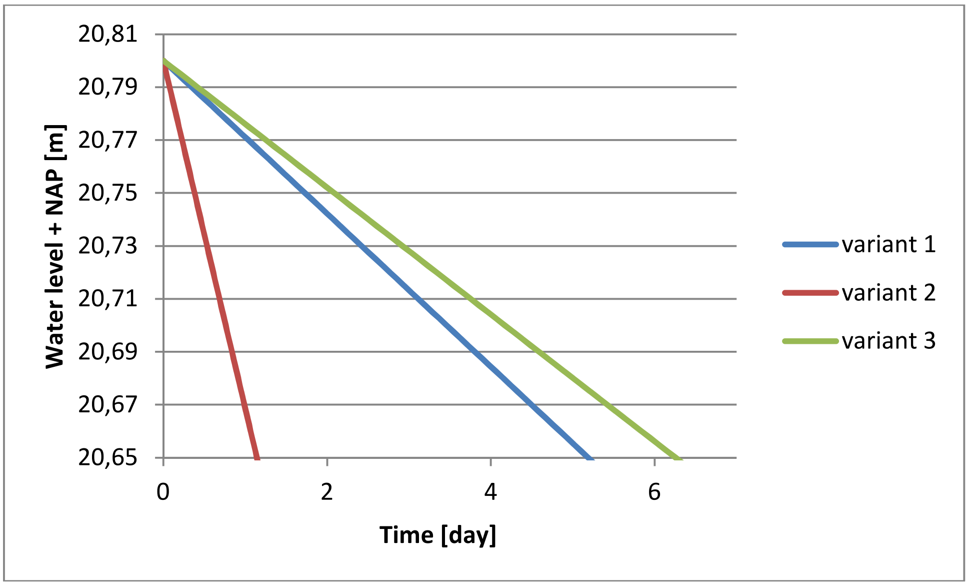
When no water shortage in the canal section arises a summation of the outflow results in a outflow of 22 m^3/s. A summation of the inflow results in a inflow of at least 24 m^3/s. To obtain a balance between the inflow and outflow 2 m^3/s should be discharged by the weir. The accuracy of 6 m^3/s of the weir is in this situation insufficient. For discharges of the Border Meuse above 13 m^3/s the accuracy of the weir is sufficient. However according to the KNMI'14 scenarios low discharges become lower and the periods of low discharges will increase in the future. To discharge the surplus of water the Stoney gates should be opened and obtaining the highest accuracy of the weir of 6 m^3/s.

Table 9 - In and outflow of canal section Linne for low discharges of the Meuse

Inflow	Type	No water shortage Q > 36 m^3/s at Maastricht St. Pieter	Water shortage Q < 36 m^3/s at Maastricht St. Pieter
		[m^3/s]	
Discharge by the Border Meuse	Variable	> 10	10
Leakage lock Maasbracht	Variable	13	9
Leakage of lock Panheel	Fixed	1	1

Outflow			
Leakage of lock Heel	Variable	7	4
Leakage of lock Linne	Fixed	3	3
Leakage weir Linne	Fixed	3	3
Pumping station Panheel	Fixed	6	4
Pumping station Maasbracht		0	7
Fish ladder of Linne	Fixed	2	2
Water extracted by the Clauscentrale.	Fixed	1	1
Discharge by hydroelectric station or/and weir	Variable	> 0	0

The analysis for low discharges is also done by means of a mass balance. In the analysis determines the effect of the water level change in the canal section due to the inaccuracy of the weir for Border Meuse discharges smaller than 13 m^3/s. The inaccuracy of the weir results that the outflow of the canal section is 4 m^3/s higher than the inflow, the result is that the water level in the canal section will drop during this situation. In the analysis a Border Meuse discharge of 10 m^3/s is assumed (the minimum required discharge over the Border Meuse). The water level is important for navigation in the canal section. When the water level becomes lower than 20.65 m +NAP navigation of fully loaded vessels is not possible anymore because the river dept becomes insufficient for the draught of class VB vessels. Only navigation for smaller vessels or not fully loaded class Vb vessels is possible. The decrease of the water level is analyzed for the three mentioned variants. The result is plotted in Graph 1. The graph represents the time that is necessary for a decrease of the water level to 20.65 m +NAP. It can be seen that the water level for de second variant (no lakes connected) reaches the water level of 20.65 m +NAP in more than 1 day. Variant 1 (current situation) reaches 20.65 m +NAP in about 5 days and variant 3 (all lakes connected) in more than 6 days. It can be concluded that the Maasplassen slow down the water level decrease with more than 4 days for variant 1 and 5 days for variant 3.



Graph 1 - Water level changes due to inaccuracy of the weir

To maintain the water level in the canal section above 20.65 m +NAP during a period of low discharges three options are possible:

- The storage area could be increased by connecting more lakes with the Meuse as corresponds with variant 3.
- The increasing of the accuracy of the weir could be included in the improvements of the weir.
- The water level in the canal section should strictly be monitored, according to the results of the monitoring the Stoney gates should frequently be lifted and lowered to maintain the water level between 20.80 m +NAP and 20.65 m +NAP.

The first option is possible however not the best option. Because it is known that the weir should be improved, the inaccuracy of the weir could be taken into account in the improvements of the weir. Both operations together (increasing the storage capacity (connecting more lakes) and improving the weir) is not an effective way of solving the problem. The last option could be possible if the current weir is totally maintained. However in this situation no improvements to the weir will be made so the insufficient situation as was presented in the problem statement (section 1.3) remains. The best option is to take the accuracy into account in improving of the weir.

3.5.3 Variant choice

The choice of the variant is based on a MCA analysis. The result of the MCA analysis is represented in Table 10. In the MCA analysis are the following aspects taken into account:

- Safety

With the factor safety is meant the safety against quick raise of water levels. Along the Meuse several villages, recreational areas and industries are located. When the water level in the canal section becomes higher, areas along the Meuse will be flooded. Municipalities or individual persons can be surprised by a quick rise of the water level. Also economical damage to industrial or personal possessions should be avoided. By a slower rise of water level is more time available to anticipate on the water level rise.

- Navigation

Navigation is an important task of the canal section because it is the main reason that weir Linne was constructed. It is possible to navigate through the canal section during high discharges. When water levels and flow velocities become high, navigation becomes difficult. During low discharges sufficient navigational depth becomes problematic. Is important that navigation on the Meuse is maintained as long as possible during high and low discharges. It is important because the economical losses should be kept as low as possible.

- Discharge

It is important that during high discharges the water can pass the canal section as quickly as possible. Obstacles in the canal section can delay the discharge of water which is not beneficial for a quick discharge.

- Adaptability

The adaptability of the variant is important to anticipate on changing conditions. Four different scenarios are distinguished based on the KNMI'14 climate change scenarios. However which of the future scenarios will become reality is not known. The it is beneficial for the variants if the variant has the possibility of adaption to a new situation.

- Industry

Industry uses water from the canal section. In most situations is the water used as cooling water. Sufficient water is required, especially during dry periods. During extreme low discharges industries are not always allowed to use water from the Meuse. Some industries have a water storage to have sufficient water during a certain time. However not all industries have such facilities, during dry periods these factories should have to decrease the production. Decreasing production has influence on the profits of the industries.

3.5.4 Rating of variants

To compare the variants is per aspect a mark granted to the variants. For every aspect are the marks 1, 2 and 3 assigned. The variant that satisfies the specific aspect the most is granted with the mark 3, the second best variant is granted with 2 points and the worst variant is marked with 1 point. At the end are the scores multiplied with weight factors because the aspects are not of equal importance. In the last line of Table 10 are the scores per variant represented.

Table 10 - MCA analysis for the variants for effect of the lakes on the water level behavior

Aspects	Weight factor	Variant 1	Variant 2	Variant 3
Safety	0.29	2	1	3
Navigation	0.19	2	1	3
Discharge	0.24	2	3	1
Adaptability	0.15	3	2	1
Industry	0.05	2	1	3
Recreation	0.10	3	1	2
Score excl. factor		14	9	13
Score incl. factor		2.29	1.65	2.18

- *Variant 1: Current situation*
  It is considered as the most preferable solution. The variant has the best performance compared the two other variants. Furthermore are the costs of this option lower than for the other variants. The combination of the high score according to the MCA analysis and the low cost prefer this variant above the other variants.
- *Variant 2: No lakes*
  It is considered as the worst preferable solution. The safety and navigable disadvantages due to the quick reaction of water levels on the discharge are not required.
- *Variant 3: All lakes*
  It is considered as a possible solution. The high scores on safety and navigation are preferable. The low scores on discharge and adaptability are not desirable. Also the costs of variant 3 are not beneficial for the variant.

3.6 Conclusion

For discharges higher than 1200 m^3/s the Meuse becomes a free flowing river, corresponding with an opened weir. For discharges higher than 1500 m^3/s the spillway is activated so that the water is not only discharged by the weir, as is represented in Table 7. Navigation is still possible for a discharges higher than 1200 m^3/s. Navigation is possible up to discharges of 2500 m^3/s which is concluded in section 3.1.3.. However vessels are not permitted to navigate over the weir when it is completely opened. The locks of Heel and Linne should be used despite the weir is opened, as was concluded in section 3.1.3.. During low discharges navigation is possible however vessels cannot be fully loaded. The draught of the vessel is decreased and navigation remains possible during low discharges. When the water level becomes lower than 20.65 m +NAP is navigation not possible for fully loaded vessels of the Vb class as can be conclude from section 3.1.3. For lower water levels, only for small vessel classes and recreational boating is possible.

The discharge accuracy of Stoney part is 6 m^3/s, which is insufficient for a minimum required discharge of 10 m^3/s at the Border Meuse. The insufficient accuracy results in a slow decrease of the water level in the canal section during discharges lower than 13 m^3/s. The Maasplassen have a certain effect on the velocity of the water level decrease due to the insufficient discharge accuracy of the weir. The presence of the Maasplassen slow down the decrease of the water level. The damping effect of the lakes is most preferable and effective during low discharges according to section 3.5.1

and section 3.5.2. Navigational dept is maintained for a longer period due to the connected lakes. The best option to improve the inaccuracy of the weir is to take the inaccuracy into account in the improvements of the weir because situation of the weir should be changed according to the problem statement. The current situation with respect to the Maasplassen could be maintained.

According to the results in section3.5.1 the damping effect has less effect on extreme high discharges compared to low discharges. Because the canal section is relatively small the water level will rise relatively quick however the lakes slow down the water level rise. The connection of extra lakes has a minor influence on this effect. Due to higher discharges in the future, the water level will rise more quickly as can be seen in Figure 21. The increasing discharge result in a quicker water level rise in the canal section. A opening procedure shorter than 8 hours is reliable after the high discharges are measured at measure point Maastricht St. Pieter, as was concluded in section 3.1.1. The duration of opening of the weir is determined by the removal of the Poirée part. The removal of the Poirée part is a manual procedure which cost a certain amount of time. An exact duration from opened to closed situation is hard to determine because the weir is not opened for a total closed condition. The measurement of flood waves at Maastricht St. Pieter is preceded by water level rises for which already partitions are removed at Linne.

Based on the conclusions for high and low discharges and the MCA analysis is concluded that the current situation of the Maasplassen could be maintained. The effect of the Maasplassen has a minor effect on the required opening time during high discharges. The Maasplassen have a certain effect on the decrease of the water level during low discharge due to the inaccuracy of the weir. However the inaccuracy of the weir will be taken into account in the improvement of the weir, presented in chapter 5.

4 Weir complex Linne

The third design level contains the pre-design of the weir structure. Before the dimensions of the weir are determined will the current weir be analyzed. It is investigated if the entire weir should be renewed or parts of the weir. In chapter 3 is concluded that the accuracy of the current weir is sufficient with respect to the low discharges. The accuracy is determined by means of the discharge over the Stoney gates. In chapter 3 is also concluded that the opening procedure of the weir should take maximal 8 hours after the flood wave is measured at measuring point Maastricht. St. Pieter. These aspects are taken into account. Also the rough dimension of the discharge cross section important conclusions of this design level because the dimensions have influence on the choice of gate type. The main criteria for the choice of gate type are based on the required width and height. After determination if the weir should be replaced or a part of the current weir should be renewed, and the rough dimensions of the discharge openings are determined, a gate type for the new situation is selected.

The structure of this chapter is comparable to the structure of the first and second chapter. The process starts with an analysis of the current weir in section 4.1. In section 4.2 are the system boundaries and the boundary conditions represented. The requirements for the pre-design are represented in section 4.3 and the functions of the weir complex are represented in section 4.4. Section 4.5 contains an analysis of the state of the current weir based on the RINK project and visits at the weir location. In section 4.6 are the required dimensions of the discharge cross section presented. The choice of the gate type for the new situation is presented in section 4.7 and the conclusion of this design level is represented in section 0.

4.1 Project area

Weir complex Linne is located on the east side of canal section Linne, as can be seen in the schematized situation of Figure 10. In Figure 22 is the weir complex of Linne represented. The weir complex consist of:

- Weir Linne
- Hydroelectric station Linne
- Fish ladder
- Spillway
- Cycling bridge (out of scope in this report)

This chapter is focusing on the weir construction. Information about the hydroelectric station can be found in chapter 3 (section 3.1.2.2, about the fish ladder in section 3.1.2.3 and about the spillway in section 3.1.2.4).

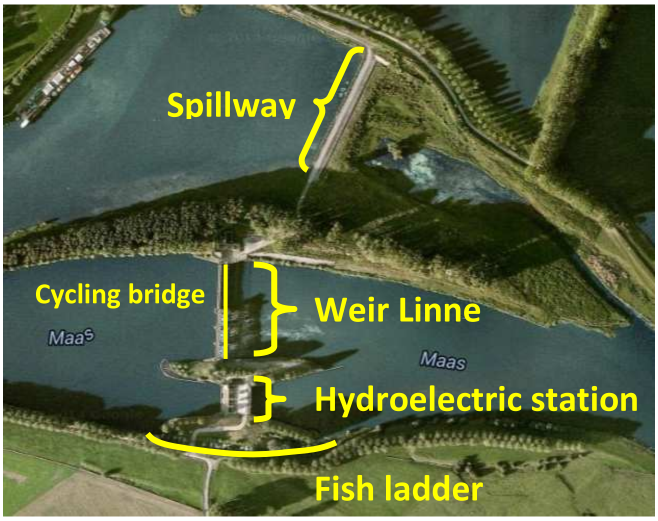


Figure 22 - Weir complex Linne (source: Google Maps)

4.1.1 Weir of Linne

Weir Linne is the second weir in the Dutch part of the Meuse. The weir of Linne is an important, high valuable architectural object and a landmark in middle Limburg. The architectural aspect is determined by the importance for history of building technology, the esthetical qualities and the innovative use of materials. For example the early and bare concrete construction is unique for hydraulic engineering in the Netherlands. The weir has an important cultural and historical value and is an example of technological development of hydraulic engineering in the Netherlands (Rijksdienst voor het Cultureel Erfgoed, 2001).

The weir construction is a combined structure and contains a Poirée weir and a Stoney part. The weir is constructed between two banks, the east bank has a height of 24.66 m +NAP and the west bank 24.00 m +NAP. The weir complex has a total width of 125 m, the discharge width is 111 m (Rijkswaterstaat, Kanalisatie van de Maas in Nederland - Verslag over de vorderingen van de werken in ther jaar 1921 en balans per 31 december 1921, 1921).

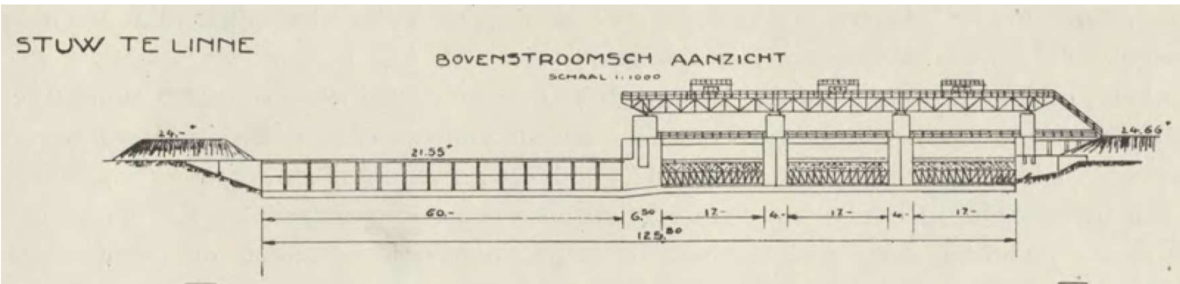


Figure 23 - Front view of the upstream part of the weir of Linne (source: Rijkswaterstaat)

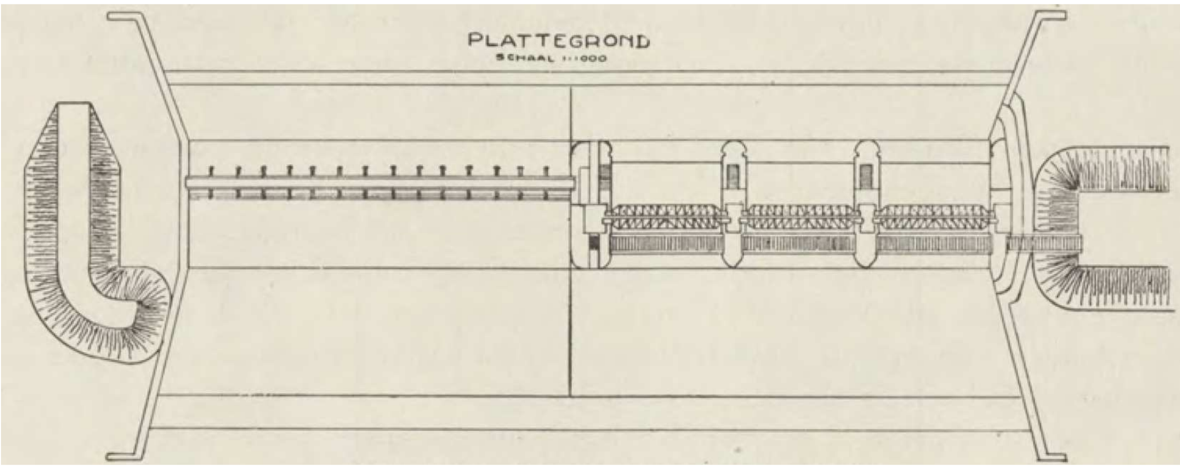


Figure 24 - Top view of the weir of Linne (source: Rijkswaterstaat)

4.1.1.1 Stoney part

The weir of Linne contains three Stoney gates of each 17 m wide. The gates are placed between four hoisting towers. The top of the hoisting towers is at 31.30 m +NAP, the towers have a height of 14.35 m. The tower between the Stoney and Poirée part has a width of 6.5 m, the other towers a width of 4.0 m. During higher discharges than 1200 m^3/s are the gates lifted from the water. The gates are lifted to a height between 24.65 m +NAP and 28.00 m +NAP. The towers are constructed on a 20 m wide reinforced concrete bottom slab. The top of the slab is located at 16.95 m +NAP and the bottom at 13.65 m +NAP. On the downstream side of the weir is a stilling pool created with a depth of 1.2 m and a length of 15 m. On both sides of the floor slab are leakage screens present. On both sides of the weir is a bottom protection present to prevent scour holes.

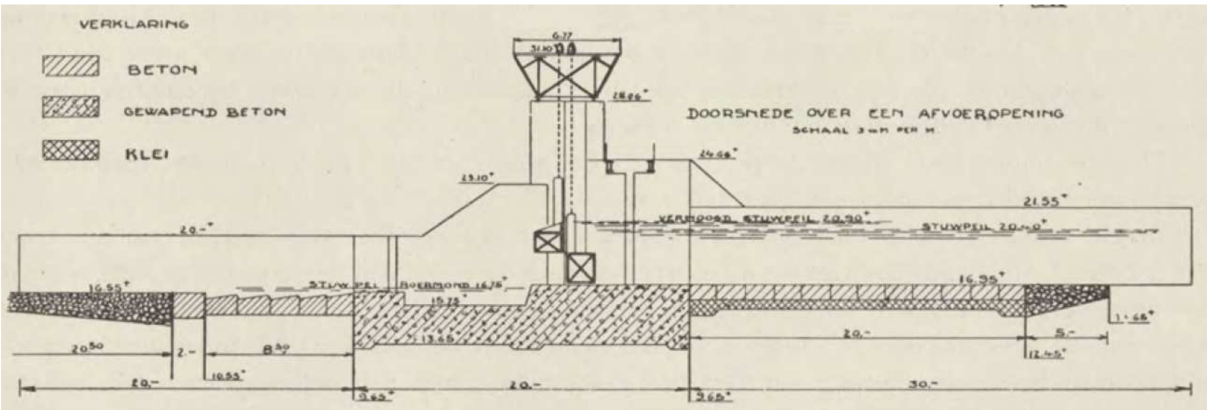


Figure 25 - Cross section of the Poirée part of weir Linne (source: Rijkswaterstaat)

The gates are inclined between two hoisting towers. A discharge opening contains one gates consisting of two gate slides, a lower and a higher slide. Both gates are lifted by separate chains which are connected to hoisting installations on top of the hoisting towers. The gates are placed in guides connected to the hoisting towers. Both gates have a separated guidance, a detail of the guides is represented in Figure 26.

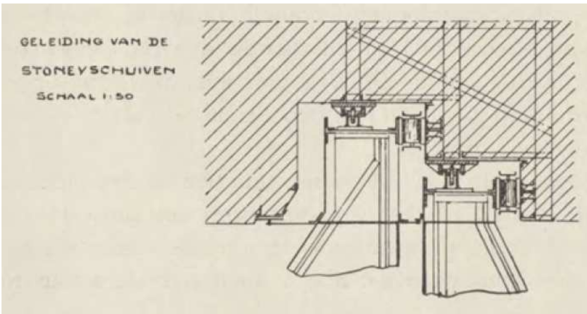


Figure 26 - Detail of the guidance of the Stoney gates (source: Rijkswaterstaat)

On the upstream and downstream sides of the weir are bottom protections present. The weir construction forms a contraction in the river cross section. The contraction results in changing flow velocities upstream and downstream of the weir. The bottom protections are represented in Figure 25.

4.1.1.2 Poirée part

The bottom of the Poirée weir consist of a thick unreinforced concrete slab. The top of the bottom slab is located at 15.95 m +NAP, the bottom plate has a thickness of 2.65 m. The slab has been founded on a shallow foundation because the subsoil consist mainly of sand and gravel. The slab has a width of 20 m. On both sides of the slab are leakage screens placed. The leakage screens have been placed until 9.05 m +NAP so have a length of 4.25 m.

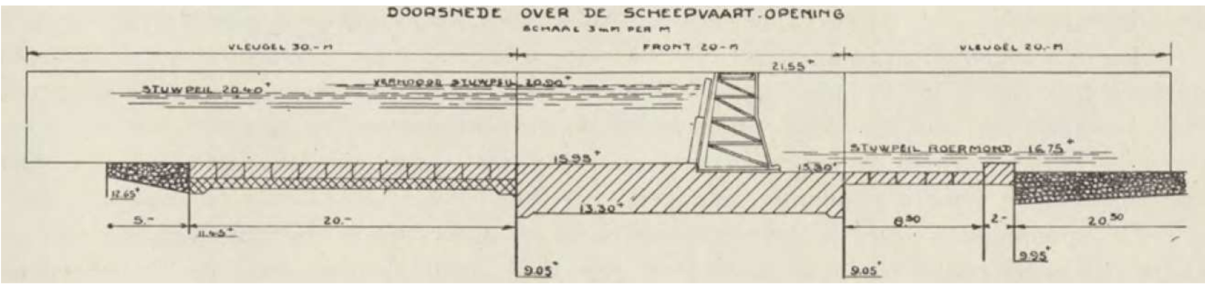


Figure 27 - Cross section of the Poirée part of weir Linne (source: Rijkswaterstaat)

The Poirée weir consists of yokes which are connected by hinges to the bottom of the weir. The distance between the yokes is 4 m. The weir contains 14 yokes, the total width of the Poirée part becomes 60 m. On the top of the jokes are demountable bridge parts placed which connect the yokes with each other. A framework between the yokes and the bridge parts is created. The top of the bridge parts are at 21.55 m + NAP. The yokes are hingedly connected to the bottom so that the yokes can rotated sideways to the bottom. The connection between the yokes and the bottom construction is represented in Figure 28. The bottom slab on the upstream side (15.95 m + NAP) of the Poirée part is 0.65 m thicker than on the downstream side (15.30 m + NAP). At the distinction between the upstream and downstream side are the hinges of the Poirée part located as can be seen in Figure 28.

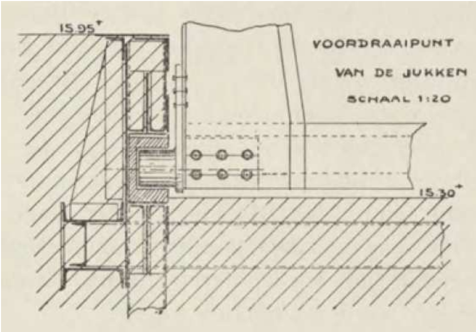


Figure 28 - Detail of the connection between yoke and bottom (source: Rijkswaterstaat)

On the upstream side the yokes side are the partitions (or plates) placed. Three partitions are placed on top of each other to obtain the required water level upstream of the weir. The partitions are made of steel and are placed or removed one-by-one by means of a crane. The amount of partitions that are removed depends on the required discharge over the weir. The crane which removes or places the partitions moves on the rail on the bridge parts.. It is theoretically possible for vessels to pass the Poirée part when Poirée gate/partitions are removed and the yokes on the bottom of the weir. However, in practice ships are required to use the locks of Linne and Heel during high discharges.

4.2 Boundaries for the pre-design of weir Linne

Section 4.2 determines the boundaries of the weir construction, the boundary conditions for the weir construction and the requirements that are important for the pre-design of the weir construction.

4.2.1 System boundaries for pre-design of weir Linne

The system boundaries of the design level spans from the approach channel at the upstream side to the approach channel at the downstream side of the weir. The system boundaries are:

- Upstream approach channel
- Upstream bottom protection
- Downstream approach channel
- Downstream bottom protection
- Weir foundation
- Super structure of the weir consist of:
    - Poirée gate
    - Stoney gates
    - Concrete structure

4.2.2 Boundary conditions

- Upstream water level
- Downstream water level

4.3 Requirements for the pre-design of weir Linne

- The discharge capacity of the weir from the upstream to the downstream boundary should be at least 1200 m^3/s (for larger discharges are the gates totally removed from the water)
- The weir meet the requirements of the ARBO legislation
- The gate should be able adaptable to required increasing water levels according to 'adaptief delta management'
- The bottom protection should be stable with respect to flow velocities behind the weir.
- The opening procedure of the weir should be shorter than 8 hours
- The weir should be able to be remote controlled

(the requirement of lifetime of is not taken into account on this design level of the weir construction because the lifetime varies by component)

Requirements obtained in design section 3.6:

- The weir should be able to respond relative quickly to high discharges
- During low discharges of the Meuse a accurate discharge control is required

4.4 Function

The main focus for the design level of the weir construction is aimed at the weir construction. The functions of the weir are:

- Regulation of upstream water levels
- Regulation of the discharge during high discharges
- Regulation of discharge during low discharges
- Transport of sediment
- Transport of ice

4.5 State of the weir

In this section is information given about RINK. In section 4.5.1 is the RINK project discussed to obtain the purpose of the RINK project. In the sections 4.5.2 is state of the concrete super structure determined and in section 4.5.3 the state of the movable parts (Stoney and Poirée gates).

4.5.1 RINK (Risico Inventaris Natte Kunstwerken)

Rijkswaterstaat manages a large number of hydraulic structures. Most structures have a lifetime of 100 years. Of the structures is 1/3 build before 1940. Rijkswaterstaat has started the RINK project to obtain a total view of the maintenance condition of hydraulic structures in relation to 'new risks' arising from changing conditions. With the term new risks is not meant new types of danger but changing aspects that have effect on current construction with respect to the applicable regulations. 'New risks' can originated from:

- Changing conditions and design principles
- Maintainability
- Usage and operability (more or other functions)
- Law and regulation.

The basis for RINK is the analysis of hydraulic structures according the RAMS systematic. The analysis has based on reliability (R), availability (A), maintainability (M) and safety (S). The main methods to obtain the four aspects are (Figure 29):

- On-site inspections
- Constructive and technical analyses
- Risk- and reliability analyses

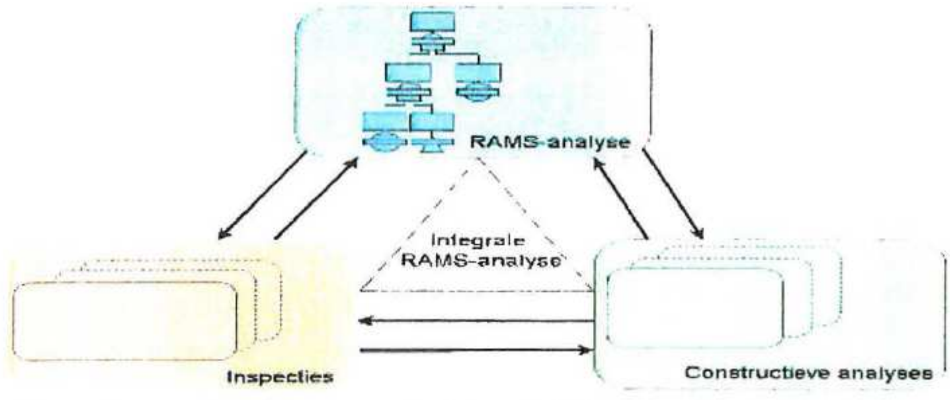


Figure 29 - Pillars of the RAMS analysis (source: Rijkswaterstaat)

Not only the single object but all objects inside a corridor are important in the RAMS analysis. To this purpose is for RINK a corridor performance model is developed. The model is a helpful tool to formulate cost-effective measure scenarios in relation to the required performance level of objects inside the corridor.

The main purposes of RINK are:

- Obtaining a overall view of the maintenance state of the hydraulic structures in relation to the new risks and determining the remaining lifetime of the structures. The maintenance state and remaining lifetime are important for the best replacement strategy.
- Determining the performance (RAMS) of the hydraulic structures based on the network level developed measuring method (SLA). On the occasion of the results are suggestions and priorities of measurements ranked to obtain a better and more cost-efficient maintenance model.
- To ensure that the RINK systematic become a fixed principle for Rijkswaterstaat whereby periodical inspections are important.

The approach for the maintenance cost of RINK is based the maintenance model represented in Figure 30. On the vertical axis are the reliability/risk levels of function losses represented. On the horizontal axis is the lifetime of the construction represented. The curves represent the condition of the construction parts. The numbers on the right side of the figure represent the different safety levels, the levels represent:

- Level 1: Loss of function
- Level 2: Intervention level to avoid loss of functions
- Level 3: Quality level of the original design
- Level 4: Quality level of the construction adapted to the "new risk"

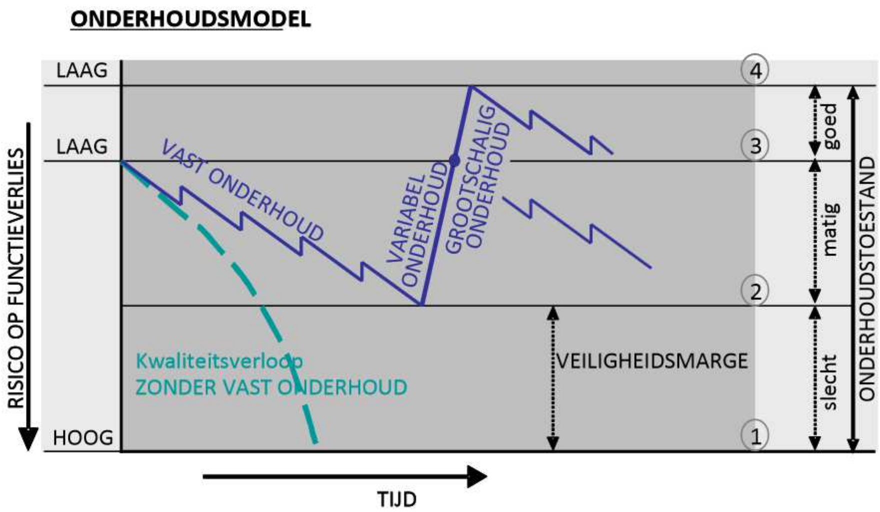


Figure 30 - Maintainance model for hydraulic constructions (source: Rijkswaterstaat)

4.5.2 Fixed structure

The fixed structure of the weir of Linne consists of four discharge openings. Three openings with a width of 17 m each (Stoney) and one opening of 60 m width (Poirée). The fixed construction is made of concrete. The concrete constructions is made between 1918 and 1921. No clear design rules were present in these times. The resulted in a strong over-dimensioned construction. The available concrete was of good quality what results in a robust fixed construction (Henk Verkerk (Rijkswaterstaat Limburg), 2014).

The concrete construction of the weir was in a bad condition according to investigations in 1993. The construction was subjected concrete degradation, large cracks and corrosion of the reinforcement were the result. The state of the pillars of the Stoney part were also in a bad condition. Vegetation in the cracks increased the velocity of the degradation process (Rijkswaterstaat, Achtergebleven onderhoud natte infrastructuur rijk, 1993). In 1997 and 2007 large reconstruction works were done to improve the concrete construction.

4.5.3 Movable parts (RINK rapport)

The RINK report for weir Linne was not available. The RINK report of weir Sambeek is used because weir Sambeek contains also a Poirée and a Stoney part. Both weirs are different but the main issues of both weirs are comparable (de Wilde, Kiljan, & Janssen, 2010).

- *Stoney part*
  The lift installation of the Stoney part can cause problem. The unavailability of the weir is closely related to failure of the lift installation. With sufficient maintenance and yearly inspections it should be possible to decrease the unavailability of the lifting installation.
- *Poirée part*
  Operation and management of the Poirée part is not safe in accordance with the ARBO legislations (=arbeidsomstandighedenwet). Two to three people are required to place or remove the partitions by hand with the aid of a small crane. The opening operation is done during high discharges over the weir. No railing is present on the Poirée part otherwise the crane is not able to move over the top of the Poirée part. An impression of the situation is given in Figure 31. Up to now this situation is tolerated however it is expected that this situation will change in the future.

Figure 31 - Unsafe situations during removal of the partitions of the Poirée part (source: youtube)

It is not advisable according to the RINK report to replace the total weir of Linne at the moment. It is advised to monitor the condition of the weir until the period 2030 - 2035. After this period a decision should be made to replace the or maintain the current weir for a longer period. This advise holds for all weirs of the Meuse corridor. Maintaining the current weir with usual maintenance interventions keeps the weir on safety level 3. Safety level 3 is a sufficient for the current situation however does not improve the performance of the weir based on the new risks. If is decided to maintain the current weir construction it is essential to restore the Poirée part for two reasons

- The weir is not safe according the ARBO legislations so measures are required to improve this situation.
- The weir is not suitable for remote operation (the Poirée part is operated by hand)

In this report is assumed that the current construction is maintained for a longer period. The Poirée part does not fulfill the requirements of the ARBO legislation and cannot be remote controlled is decided to design a new gate. This gate will be constructed for the Poirée part of the weir. A second reason to chose for the design of a new gate for the current instruction is also made because a net weir design for a Meuse weir is already done.

4.6 Dimensions of the discharge width

In this section is a rough determination given about the discharge width. The discharge width of the of the current weir (and the Poirée part) is determined to check if the discharge capacity of the weir is sufficient for another weir gate and to determined if the discharge width is sufficient with respect to the bottom protection behind the weir. In this section the required width of the weir and the required retaining height of the weir are determined. The current weir construction has a certain width, in this chapter is checked if the width of the weir could be increased. An smaller weir width could result in extra column on the Poirée part. With extra columns the number of possible gate types that could be used increases. The number of gates increases because with extra columns also gates that are not able to span the 60 m of the current Poirée part could taken into account. The width of the weir is checked according the stability of the bottom protection behind the weir (influenced by the flow velocity over a opened weir) and the discharge capacity of the current weir.

4.6.1 Discharge type

Discharges over gates fit into two discharge classifications, namely:

- *Overflow gates*

At overflow gates is water discharged over the crest of the weir gates. The overflow gate is used for two purposes namely:
  - Water level regulation
  - Discharge of floating material
- *Underflow gates*
  At underflow gates is the water discharged by an opening between the weir bottom and underside of the gate. The overflow gate is used for two purposes namely:
  - Discharge regulation
  - Sediment transport

The Stoney part is able to create a underflow and an overflow. Discharge regulation during low discharge and part of the sediment transport can be done with the Stoney part. The sediment in front of the Poirée part is removed during high discharges, when the Poirée part is removed from the water. The Poirée gates should satisfy the function of water level regulation during low discharges and discharge during high discharges. At the Poirée part a new gate type with overflow should satisfy.

4.6.2 Width

The width of the weir is an important factor for the discharge capacity of the weir. The weir forms a constrain in the water way. An insufficient discharge width results during high discharges in increasing water level rises upstream of the weir. High discharges result in high water levels and high flow velocities behind the weir, these can damage the bottom protection behind the weir. The required width of the weir is analyzed in two ways:

- Maximum flow velocity behind the weir
- Required discharge capacity of the weir

The discharge width of the weir (Poirée part) could have influence on the choice of a new gate. A increasing discharge width of the Poirée part decreases the choice of a suitable gate type. The gate type is determined in section 4.7.

Measurements should be taken on the Poirée part according to section 4.5.3. In the analysis of the width is the Stoney part assumed to be constant (3 x 17 m), the Poirée part is assumed variable. The values used in the model are represented in Table 11. The water levels upstream and the width of the upstream side of the weir originate from Figure 6 and Table 3.

Table 11 - Values used for determination of the weir width

Description	Symbol	Value
Upstream water level	h_{up}	4.85 m +NAP*
Discharge	Q	variable
Gravitational acceleration	g	9.81 m/s^2
Upstream width	B_{Meuse} (=B_{down} = B_{up})	120 m

** the upstream water level is the result of the difference between the water level in canal section Linne and the bottom level of the weir (20.80 m + NAP - 15.95 m + NAP = 4.85 m)*

4.6.2.1 Velocity behind the weir

The weir is (also for a opened weir with full discharge capacity) a constriction in the river profile. The constriction results in an increase of the flow velocity. High flow velocities can have influence on the stability of the bottom protection of the weir. In this report the focus is on the downstream bottom protection. A instable bottom protection may result in damage to the protection and possible instability of the weir construction.

The width of the weir is related to the discharge capacity of the weir. The required design flow velocity behind the weir is not totally clear. It is questionable if the bottom protections remains stable for discharges higher than 3 m/s during a bank full discharges of 3453 m^3/s (van Vuren, Leeuwdrent, & Vieira da Silva, 2009). In this report is assumed that discharges behind the weir should not be higher than 2.5 - 3 m/s. Higher discharges can affect the bottom protection behind the weir. Damage to the bottom protection should be avoided because the damage is not visible from the water surface. A damaged bottom protection results in an increasing probability of constructional instability. The used model is based on two principles as represented in Figure 32.

- Preservation of the energy head just in front and just behind the weir

$$H_{up} = H_{down}$$

$$h_{up} + \frac{u_{up}^2}{2g} = h_{down} + \frac{u_{down}^2}{2g}$$

Equation 5 - Preservation of the energy head

- Preservation of discharge upstream and downstream of the weir

$$Q_{up} = Q_{down}$$

$$B_{up} h_{up} u_{up} = B_{down} h_{down} u_{down}$$

Equation 6 - Preservation of discharge

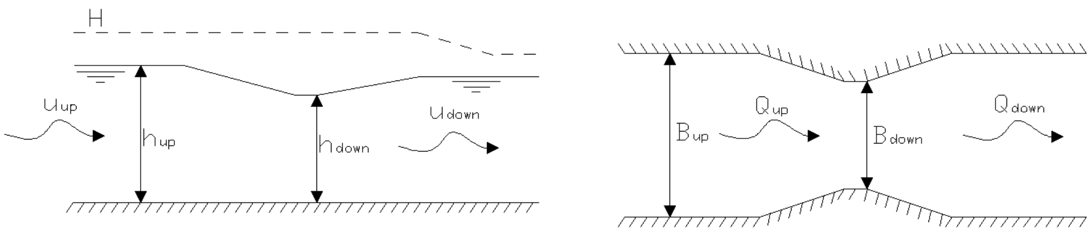


Figure 32 - Left: preservation of the energy head; Right: preservation of discharge (top view)

By means of Equation 5 and Equation 6 can the flow velocity behind the weir be calculated. The progress of the flow velocity behind the weir with respect to the discharge is represented in Figure 33. The total discharge width of the current weir is 111 m (Poiree 60 m; Stoney 3x17 m), the flow velocity behind the weir is less than 2.3 m/s at the moment the weir is completely opened (at 1200 m^3/s). According to the figure does the flow velocities behind the weir not affect the bottom protection at the moment the weir is just completely opened. The flow velocity of full bank discharge is about 3 m^3/s (van Vuren, Leeuwdrent, & Vieira da Silva, 2009).

It has to be mentioned that the flow velocities presented in Figure 33 are the flow velocities just behind the weir. The bottom protection is not directly located behind the weir gates but at a distance from the gates. The actual flow velocity at the bottom protection will be smaller due to energy losses. The energy losses are the result of eddies in the water flow between gate and bottom protection.

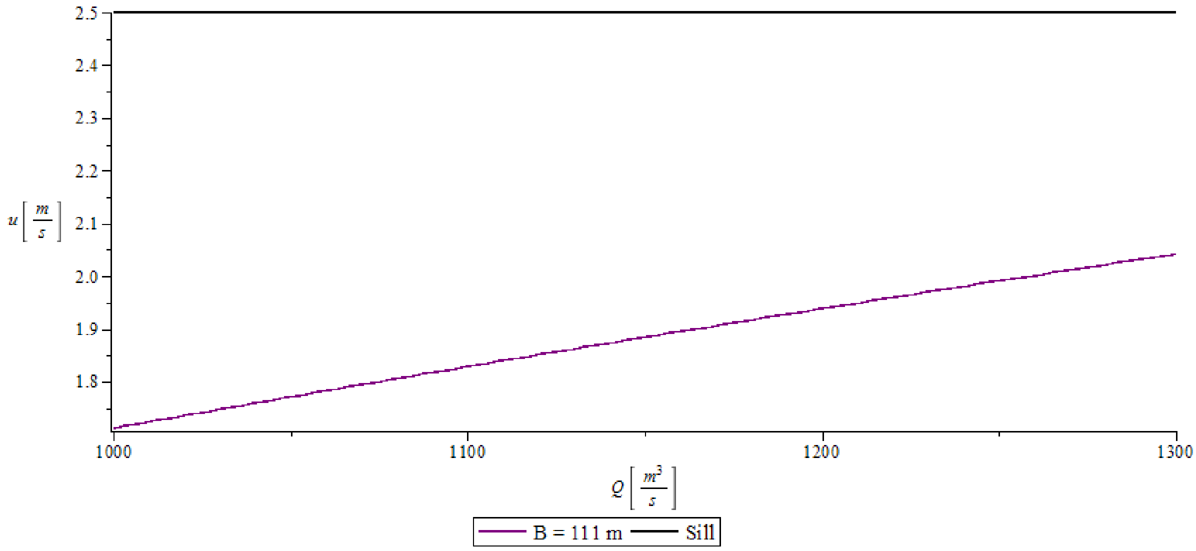


Figure 33 - Flow velocities behind the weir during high discharges

In Figure 34 are flow velocities represented for different widths of the weir. On the horizontal axis is the discharge presented and on the vertical axis the corresponding flow velocities. It can be seen that widths up to 91 m meet the requirement for the flow velocity. Based on the maximum flow velocity is a width of 91 m or larger is sufficient, this results in a minimum width of 40 m for the Poirée part. More information is presented in Appendix H.

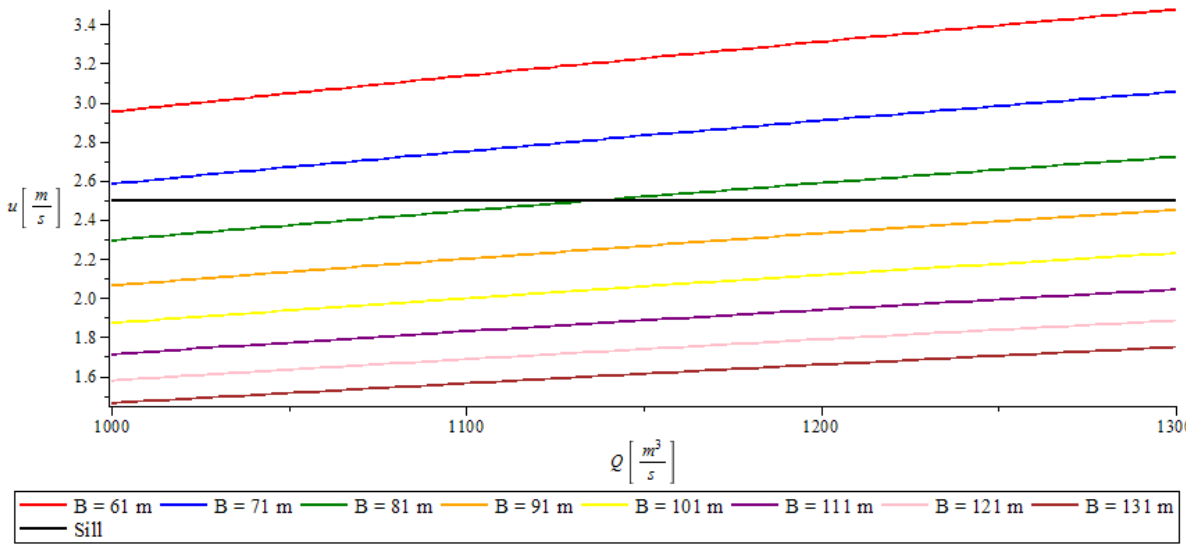


Figure 34 - Flow velocities behind the weir during high discharges for different widths of the weir.

4.6.2.2 Discharge capacity and width of the weir

The weir should be able to discharge 1200m^3/s when it is completely opened. The discharge capacity of the weir is determined for a weir with over flow. Two types of overflow weirs are known namely:

- *Perfect weir (structure controlled flow)*
  The discharge of the perfect weir is only determined by the upstream water level, width of the discharge opening and the crest height. The discharge of the perfect weir is represented in Equation 7.

$$Q = m\frac{2}{3}(h_{up} - h_{cr})B\sqrt{\frac{2}{3}g(h_{up} - h_{cr})}$$

Equation 7 - Discharge of the perfect weir

- Imperfect weir (tail controlled flow)
  The discharge of the imperfect weir is besides the discharge openings, the upstream water level and the crest height also determined by the downstream water level. The discharge of the perfect weir is represented in Equation 8.

$$Q = m(h_{down} - h_{cr})B\sqrt{2g(h_{up} - h_{down})}$$

Equation 8 - Discharge of the imperfect weir

Equation 7 seems a different compared to the equation that is usually used in literature. The reason is that Equation 7 is multiplied with Equation 9. This equation represents a relation between the discharge coefficients of a perfect weir and an imperfect weir. The advantage of Equation 9 in combination with Equation 7 is that one discharge coefficient for the perfect and imperfect weir is used (Nortier, 1989). The value of the discharge coefficient is 0.685 for the weir of Linne (van Knippenberg, 1978).

$$m_{pw} = \frac{2}{3}\sqrt{\frac{2}{3} * g m_{pi}}$$

Equation 9 - Relation of discharge coefficients of a perfect and imperfect weir

A movable weir contains both discharge patterns. The first discharge pattern is the perfect weir. The height difference between upstream and downstream water level is relatively large for a closed weir. The difference for the weir Linne is for this situation 3.95 m. Due to the lowering of the crest height and increasing downstream water level, becomes the head difference smaller until the downstream water level becomes an influencing factor on the discharge. The discharge pattern becomes imperfect. There exist a transition point between the perfect and imperfect weir situation. A perfect weir situation becomes an imperfect weir situation when the downstream water level influences the flow over the weir or in other words the upstream water level. The transition point is represented by Equation 10.

$$h_{down} = \frac{1}{3} * (2h_{up} - h_{cr})$$

Equation 10 - Transition point form perfect to imperfect weir

It has to be investigated whether the discharge equations and the crest height equations for a perfect and imperfect weir flow are differentiable. The equations are differentiable when the graphs intersect at the same angle so that a continuous transition from perfect to imperfect weir is created. The progress of the crest height with respect to the discharge over the weir is represented in Figure 35, on the vertical axis the crest height of the weir gate is presented and on the horizontal axis the discharge over the weir. It can be seen that the transition point between the perfect and imperfect flow pattern is around 970 m^3/s for a weir crest of 17.15 m. The discharge decreases rapidly when the flow pattern is changed from perfect to imperfect, the downstream water level affects the discharge capacity.

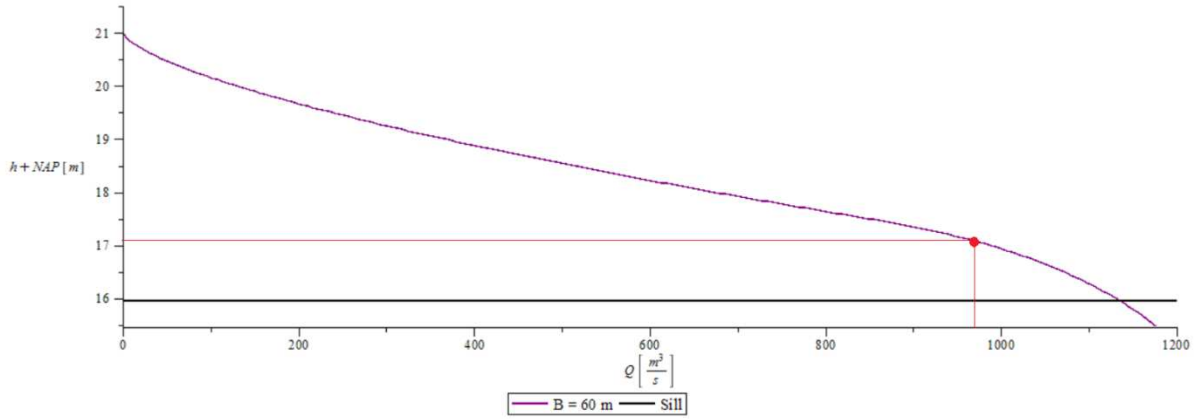


Figure 35 - Crest height for increasing discharges

It is remarkable that according to Figure 35 the weir is opened but the required discharge of 1200 m^3/s cannot be obtained. This can be explained by analyzing the downstream boundary, the water level of canal section Roermond. The downstream water level rise is modeled for a waterway with a constant width. However, canal section Roermond contains just like canal section Linne several lakes. These lakes have a damping effect on the water level of canal section Roermond. The real water level rise will become smaller than the modeled water level. The downstream water level is modeled according to Equation 11. For a larger storage capacity of the water level will the water level rise become smaller and results in a higher transition point between perfect and imperfect.

$$h_{down} = \left(\frac{Q^2}{B_{Meuse}^2 * C^2 * i_b}\right)^{1/3}$$

Equation 11 - Downstream water level

The flow characteristics for other widths of the Poirée are determined in order to investigate the effects of changing widths of the Poirée part. The characteristics are represented in Figure 36, also the sill of the weir is represented in the graph. Widths from 10 to 80 m are used in steps of 10 m. The discharge capacity m in Equation 7 and Equation 8 is usually valued between 0.7 and 1.4. For a new weir a discharge coefficient of 0.8 is used instead of 0.685 for the current weir (including Poirée part).

On the vertical axis of Figure 36 the crest height of the weir gate is presented and on the horizontal axis the discharge over the weir. The figure represents that for larger widths the discharge capacity increases. However the larger the width, the smaller the increase of capacity. It can also be seen that the width of the Poirée part could probably be decreased to 50 m. Further investigation should be done to the possibilities of minimization of the Poirée part. The advantage could be that a extra pillar on the Poirée part could be possible. This could be necessary if the new gate type is not able to span more than 50 m. A disadvantage is the quicker raise of water level in canal section Linne during high discharges.

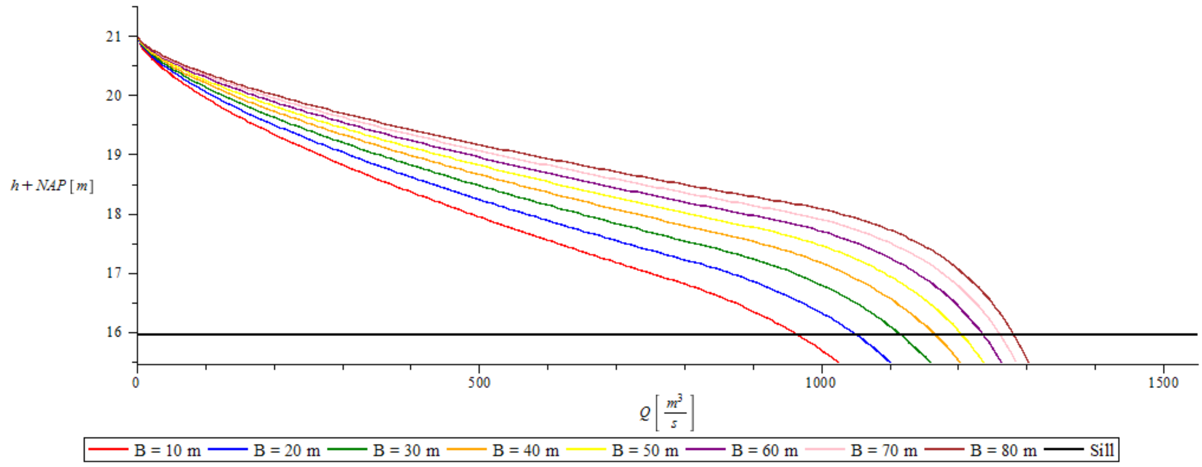


Figure 36 - Crest height for increasing discharges during different discharge widths

However the width of the Poirée part could possibly be reduced is decided to maintain the width of the current Poirée part, the current width is sufficient for the current discharges. So in the further chapters of this report is a width of 60 m for the Poirée part and 3 x 17 m for the Stoney part used.

For higher discharges in the future a larger discharge capacity could be required. Several options are possible. Possible options to increase the discharge capacity are:

- *Higher upstream water level*
  A higher upstream water level increases the height difference between the upstream and downstream water level of the weir. A higher water level difference results in a higher transition discharge between perfect and imperfect flow pattern. The lines of Figure 36 are shifted to the right.
- *Larger width of the weir*
  By changing the width of the weir, the discharge capacity decreases as can be seen in Equation 7 and Equation 8. The factor width is included in both equations so has effect on the perfect and the imperfect weir pattern.
- *Larger storing capacity downstream*
  A lager storage capacity downstream results in a slower rise of the downstream water level. The result is that the perfect discharge situation is maintained for a longer period because the downstream water level does not influence the discharge pattern.
- *Larger bottom gradient downstream*
  A larger bottom gradient downstream results in higher flow velocities in the downstream river part.

4.6.3 Height of the gate

The maintained water level is maintained by the height of the weir. For the moment is no need to increase the height of the weir gates. The water level of 20.80 m +NAP is sufficient for navigation in the canal section. However a water level of 21.00 m +NAP is assumed. A water level rise in the future could become necessary for two reasons namely:

- *Larger vessels on the Meuse*
  The demand for larger vessels is determined by the economical situation and the population growth of the Netherland. The development of these aspects are unknown at these days. The KNMI'14 scenarios "Stroom" and "Druk" take the aspects into account. The KNMI'14 scenarios are stated in Appendix D.
- *Higher discharge over the weir*
  A higher upstream water level increases the height difference over the weir. A higher height difference result in a larger discharge capacity by the weir The reason is explained under 4.6.2.2.

4.6.4 Results

In Table 12 are the results of section 4.6 represented as summary of this chapter. The rough dimensions of the gate are used in the design of the gate in chapter 5.

Table 12 - Results for the rough dimensions of the new weir gate

Description	Symbol	Value
Upstream water level	h_{up}	5.05 m*
Width of the Poirée part	$B_{Poirée}$	60 m
Discharge when completely opened	Q	1200 m^3/s

** the upstream water level is the result of the difference between the water level in canal section Linne and the bottom level of the weir (21.00 m + NAP - 15.95 m + NAP = 5.05 m)*

4.7 Gate types

In this section a new gate is selected for the Poirée part of weir Linne. The different types of closing elements are represented in Appendix A. The selection for a new gate is made in three steps. In the first step are the main properties of a closing element (overflow) investigated and the importance of the certain properties. Closing elements that not met the required properties are dropped out of the selection. In the second step are the remaining closing elements compared to important performance aspects like maintainability, material usage and safety. The three most preferable closing elements are taken to the third step. The third step contains the MCA analysis. In this step is the final type of closing element determined. The closing elements that are considered are:

- Sector gate
- Inflatable construction
- Flap gate
- Radial gate
- Roller gates
- Poirée segments
- Visor gate
- Lifting gates (Sliding gates, Wheel gates or Stoney gates)

4.7.1 Step 1: Basic properties of a closing element

In Table 13 are the main properties and the preferable way of performance represented. The first inventory of the closing element is made by means of four properties namely:

- Retaining water
- Movability
- Number of elements
- Managing of water discharge

For every aspect the most preferable property and the importance of the property appointed. All of the selected closing elements meet the preferable properties for the new closing element. All the elements are going to the second step.

Table 13 - Required basic properties of a closing element

	Preferable	Importance	Remark
Retaining water			
• **Direction load transport**	downward / sideward*	middle	*Upward is possible however lifting towers should be placed
• **Behavior of closing element**	stiff	low	

Movability			
• **Direction**	translation in y-direction / rotation around the y-axis	middle	
Number of elements	single element*	high	*More elements are possible however extra columns are necessary.
Managing of water discharge	Overflow*	high	*An underflow can be obtained by lifting the lower Stoney gate.

4.7.2 Step 2: Performance aspects of a closing element

The performance aspects are important in the choice for a suitable closing element. The general properties of the selected closing elements (van der Ziel & Dijk, 2009) are tested to the requirements for the new closing element. The performance aspects that are used to compare the closing elements are:

- Maximum width
- Maximum height
- Maximum retaining height
- Modular expandable
- Controlling of the water level
- Material usage
- Probability of failure
- Maintenance

An inflatable construction could be a preferable option. However, it is not common to use an inflatable construction like the Ramspol barrier as river weir. The inflatable construction is mainly used as barrier. An inflatable construction in combined with flap gate is on the other hand a suitable as river weir. The consideration of gate types in the first step is represented in Appendix I. The three most preferable closing elements are:

- Radial gate
- Lifting gate
- Inflatable flap gate

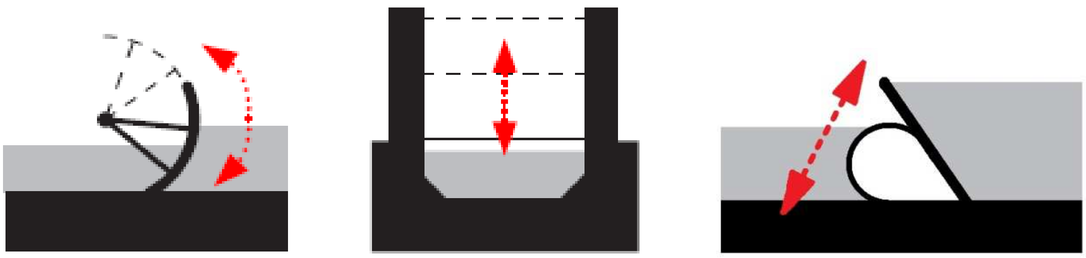


Figure 37 - Considered gate types, from left to right: radial gate, lifting gate, inflatable flap gate

4.7.3 Step 3 - Multi criteria analysis

The MCA analysis is made for the three remaining gates. The aspects that are taken into account are:

- *Safety*
  The construction should be safe, the probability of failure should be low. Failure of the weir results in an obstruction for navigational traffic.
- *Adaptability*
  The height construction should be adaptable. If in the future larger vessels are required on the Meuse should the construction be adaptable to obtain a higher water level in the canal section.
- *Maintenance*

Less maintenance has a positive effect on the choice of gate type. The maintenance aspect focuses on the construction instead of the materials. For example, maintenance becomes more difficult when important parts of the construction are under water.

- *Construction time*
  Construction time is an important aspect. The gate (or parts of the gate) should be placed between two high discharge periods. Periods of high discharges are between December and February.

Table 14 represents the result of the MCA analysis for the new gate type. For every aspect are 3 points assigned to the gate type that is most beneficial to the aspect, 2 points to the second best and 1 point to the worst option. The most important aspect is according to the table the aspect safety. The inflatable flap gate has obtained 3 points because this gate can be carried out in a required number of gate parts. The number of radial and lifting gates is less free to chose because these gates require extra columns for load transport to the foundation. The minimum discharge width of the Poirée part should be according to section 4.6.2.2 50 m. Extra columns obstruct the discharge opening, the result is a dependant number of columns so a dependant number of gate elements for the radial and lifting gates.

Based on the MCA analysis and to avoid extra columns to maintain the current discharge capacity is advisable to use the inflatable flap gate. More information about the flap gate is represented in chapter 5.

Table 14 - MCA analysis for the new gate type of weir Linne

Aspects	Weight factor	Radial gate	Lifting gate	Inflatable flap gate
Safety	0.4	2	1	3
Adaptability	0.1	1	2	3
Maintenace	0.3	2	3	1
Construction time	0.1	2	1	3
Discharge regulation	0.1	1	3	2
Score excl. factor		8	10	12
Score incl. factor		1.8	1.9	2.3

4.7.4 Behavior of the inflatable flap gate

Flap gates are hinged along the upstream edge of the gate and attached to a sill foundation. They are stored submerged and flat to the bottom. To close the flow, the downstream edge is rotated upward due to a inflatable bladder on the downstream side of the gate. The bladder is filled and pushed the gate upwards, by emptying the bladder is the gated rotated to the weir bottom. A flap gate fulfills (when properly designed) the ARBO legislations and is able to be remote controlled. More information about inflatable flap gates is presented in chapter 5.

4.8 Conclusion

It is not advisable according to the RINK report to replace the total weir of Linne at the moment. It is advised to monitor the condition of the weir until the period 2030 - 2035. After this period a decision should be made to replace the or maintain the current weir for a longer period. As presented in section 4.5.3 is decided for this report to maintain the current weir construction. In section 4.5.2 is presented the concrete super construction a robust construction due to over dimensioning. Due to the over dimensioning could the construction fulfill for longer lifetime. However it has to be investigated if the bad state of the concrete construction in the past (due to bad maintenance) has effect on the robustness of the construction. It is decided according to section 4.5.3 to construct a new weir on the old Poirée part. The Poirée gate does not meet the requirements according to the ARBO legislations and the gate is not able to be remote controlled.

The width of the Poirée part is tested in section 4.6.2 based on the discharge capacity of 1200 m^3/s. The width is checked to obtain if the width of weir is sufficient when the weir is fully opened and if the flow velocities behind the weir are do not affect the bottom protection downstream of the weir. As is concluded in section 4.6.2.1 and 4.6.2.2 is the width of the Poirée part of 30 sufficient. The width could even possibly reduced if required. More investigation is necessary to investigate if a smaller width of the Poirée part is possible.
The upstream water level for the new weir gate is assumed at 21.00 m +NAP. According to section 4.6.3 is a raise of the water level from 20.80 m +NAP to 21.00 m +NAP not necessary for the moment. However, when KNMI'14 scenarios "Stroom" and "Druk" are taken into account, the probability arises that in the future larger vessels will navigate on the Meuse. Larger vessels result in larger navigable depts.

Section 4.7 is based on a MCA analysis a new weir gate selected. The new type of weir that is chose to replace the old Poirée gates is the inflatable flap gate. The flap gate is able to retain a water level of 21.00 m + NAP and able to close the discharge width of 60 m. A flap gate fulfills (when properly designed) the the ARBO legislations and is able to be remote controlled.

5 Design of the inflatable flap gate

The fifth chapter contains the first design of the inflatable flap gate. The choice for the flap gate was made in chapter 4. The gate will be designed in high strength concrete because it is a durable material and the material properties are well known. The design of the construction will mainly focus on the design of the gate, to investigate if a concrete flap gate is possible as replacement of the Poirée gates of weir Linne. Lock gates out of high strength concrete have been applied at lock 124 at IJburg (in Amsterdam). However a concrete weir gate has never been constructed. A detailed design of the bladder is out of scope in this report, only a simple check of the strength of the bladder is taken into account. However the design aspects for the bladder like the type of filling are taken into account while the bladder itself will not be calculated. The gate is calculated based on the loads on the gate at the location of weir Linne. The concrete gate will be checked on minimum and maximum reinforcement and cracking. The design of the gate could be optimized, not only in load reduction but also to increase the discharge coefficient of the weir. By choosing a good weir shape the discharge capacity of the weir could be increased.

The flap gate is placed on the Poirée part of the weir. According to section 2.2.1 the discharge of the Meuse varies during the year. For the placing of the inflatable flap gate are the periods of high and low discharge of importance. The way of connecting the flap gate to the bottom construction is important, the support loads of the flap gate should not be to large because otherwise damage to the concrete sub structure could possibly occur.

In section 5.1 general information is represented about inflatable constructions and inflatable flap gates. The functions of the flap gate are described in section 5.2. In section 5.3 the boundaries and the boundary conditions of the flap gate are represented. The requirements to the first design of the inflatable flap gate are described in section 5.4. In section 5.5 design choices are represented that are important in calculation of the gate. The loads acting on the weir gate are presented in section 5.6. The moment and shear load distribution on the gate are presented in section 5.7. Section 5.8 determines the reinforcement and checks the gate on cracks and failure. Optimization possibility with respect weight reduction and discharge capacity of the gate is discussed in section 5.9. Section 5.10 determines the aspects for the placement of the weir on the Poirée part of the weir. The conclusion of chapter 5 is presented in section 5.12.

5.1 Inflatable flap gate

An inflatable barrier consists of different elements, namely the gate which has to retain the water on the upstream side, a membrane that will keep the gate in raised position, a filling system that transports the water/air to the membrane and a chain to prevent bouncing of the gate on the bladder. Usually the membrane is connected to a concrete sub structure, however other solutions are possible. Figure 38 presents a cross section of a inflatable flap gate, the gate, the bladder and chain are clearly visible. In this report the bellow is called the bladder (= blaas) of the weir, as is done in Figure 38. The filling system is not represented in the figure because the pumps of the filling system are normally located besides the weir construction.

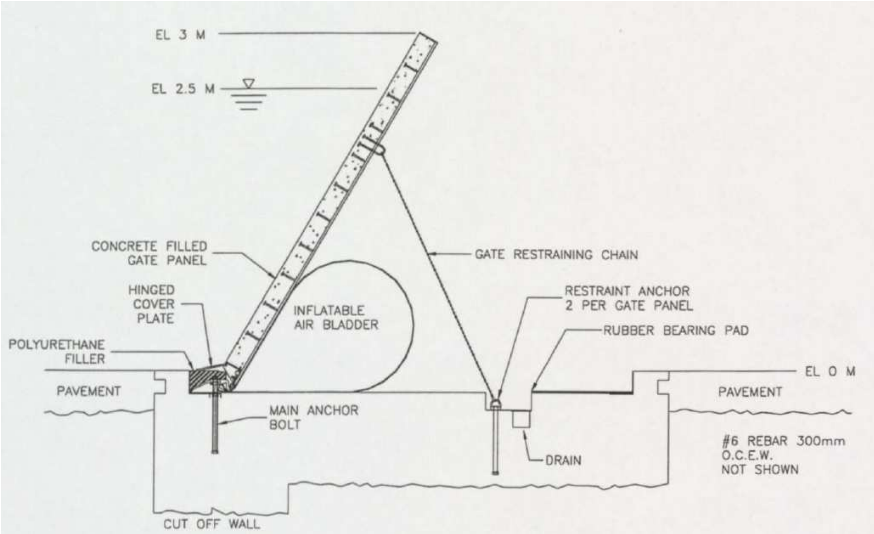


Figure 38 - Cross section of the inflatable flap gate (Source: Obermeyer hydro inc.)

5.1.1 Types of inflatable constructions

Inflatable weirs can be divided in two different types: the self retaining type and the supporting type. The self retaining type is different from the supporting type however the self retaining type gives a clear view about the behavior of the bladder.

5.1.1.1 Self retaining types

The self retaining inflatable weir forms in inflated situation a water tight construction. The weir consists of a double folded membrane which is at one or two points connected to the foundation. In the one-side connected construction is the membrane in cross sectional direction connected at one-side to the foundation, as is represented in Figure 39. The connection points are also located on the abutments and/or piers to create an impermeable weir. After filling of the double folded membrane (with water and/or air) the membrane will close the gap between the abutments and/or piers.

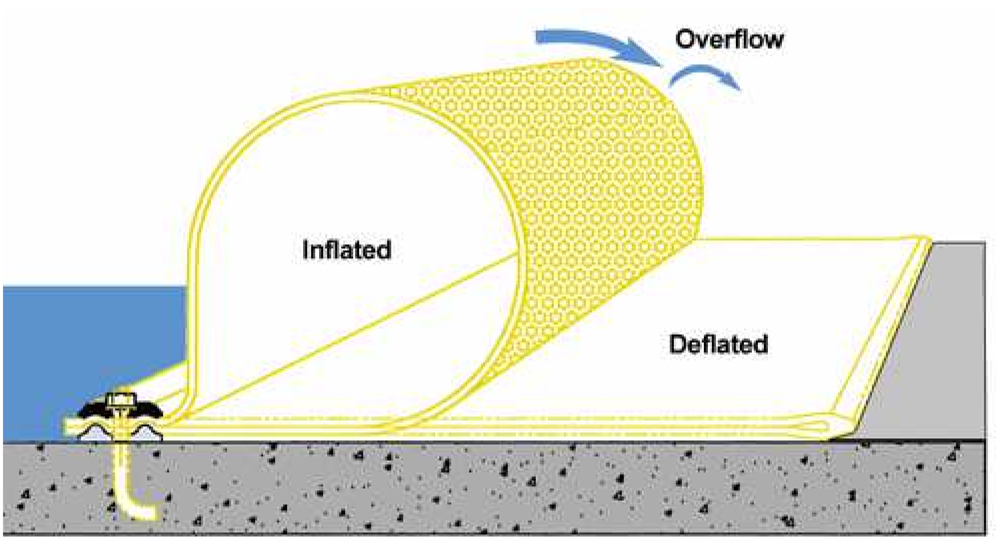


Figure 39 - One-side connected membrane (Rijkswaterstaat & WL| Delft Hydraulics, 2005)

In the double-side connected alternative are both sides of the membrane in cross section separately connected to the foundation. Together with the foundation the membrane will form a closed construction which can be filled with water and/or air. An example of the alternative is represented in Figure 40. The advantage of this alternative compared to the one-side connected type are:

- the smaller membrane length for the same retaining height
- the better option of maintenance
- the construction can fulfill a two-side retaining function

The self retaining type is usually used as storm surge barrier because the construction could fulfill its double side retaining function.

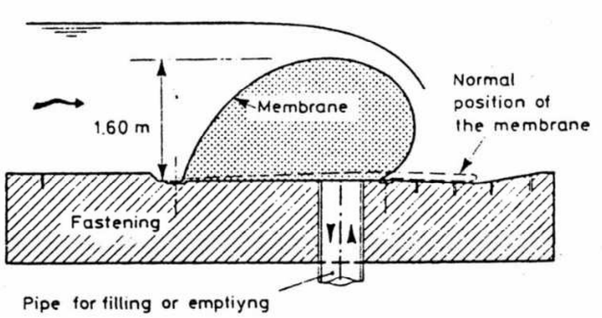


Figure 40 - Two-side connected membrane (Rijkswaterstaat & WL| Delft Hydraulics, 2005)

5.1.1.2 Supporting type

In the supporting type is the filled membrane combined with a at the bottom hinged flap. The filled membrane has the function to pushing the flap upwards and support the flap over the whole length when the flap has reached the required height. The filled membrane acts as drive and supporting construction for the retaining flap. In his type of construction the membrane can be carried out smaller than for the self retaining type. The membrane can be one-side or double-side connected to the bottom floor. Also variants of multiple small membranes are sometimes used. The flap retains the water level on the upstream side. The flap is connected by for example rubber strips to the abutments and/or piers. This type of inflatable weir is usually used in rivers because the high water side is here always on the same side of the weir.

5.1.1.3 Obermeyer gate (Obermeyer hydro inc.)

The Obermeyer gate is a well know example of a supporting type. The Obermayer gate is designed in America by Obermeyer Hydro Inc. The Obermeyer spillway gate system is a row of gate panels supported on their downstream side by inflatable air bladders. By controlling the pressure in the bladders, the crest height of the gates can be infinitely adjusted within the system control range (full inflation to full deflation). The spillway gate system is attached to the foundation by stainless steel anchor bolts. The required number of bladders are clamped to the anchor bolts and connected to the air supply pipes. The Obermeyer spillway gates are custom designed conform to any existing or desired spillway cross-section. Obermeyer spillway gates have been installed at a number of projects across the world, including Ophua dam in New Zealand, Friant and Horseshoe dams in the US, and Rarik dam in Iceland. The *Obermeyer* Spillway Gate system has many unique attributes that include:

- Accurate automatic pond level control even under power failure conditions.
- Modular design that simplifies installation and maintenance.
- Unlike hydraulic spillway gates, Obermeyer gates are supported for their entire width by an inflatable air bladder, resulting in simple foundation requirements and a cost effective, efficient gate structure.
- Thin profile efficiently passes flood flows, ice and debris.
- No intermediate piers are required.
- Obermeyer Spillway gates are a great investment due to increased revenue, decreased maintenance, and low cost of installation.

A cross section of the inflatable flap gate is represented in Figure 38. The four main elements of the inflatable flap gate are:

- *Inflatable air bladders (terminology according to Obermeyer hydro inc.)*
  The inflatable bladders are located on the downstream part of the weir. The bladders support the gate and keep the gate in raised position. The bladders lift or lower the gate by filling the bladders with air. Usually more than one bladder is used, the supporting bladders can be placed in two manners, next to each other and upon each other. Usually more bladders next to each other in longitudinal direction are used. The reason is that if one of the bladder fails the gate remains in raised position. Bladders upon each other can be used to shift the pressure point from the hinged connection to the free top end, a representation of this situation is presented in Figure 43.
- *Gate*
  The gate retains the water on the upstream side and manages the discharge over the weir. The gate is supported by bladders on the underside of the gate. The upper part of the gate forms a cantilever construction
- *Air system (not presented in Figure 38)*
  One or more pumps are located near the weir construction. The pumps will fill or empty the bladders. The air is transported by tubes, the tubes are mainly located in the bottom of the weir construction.
- *Chains*
  Chains are narrow straps located on the downstream side of the weir gate as is represented in Figure 38. The chains prevent bouncing of the gate on the bladder induced by wave loads For fully raised gates the chains are in tension, the tension in the chains result in a (pre)tension in the gate.

Figure 41 represents the use of the Obermeyer system in real life. On the downstream side are the bladders, chains and the gate clearly visible. It can be seen that the gate is not supported by one long single bladder but by multiple smaller bladders.

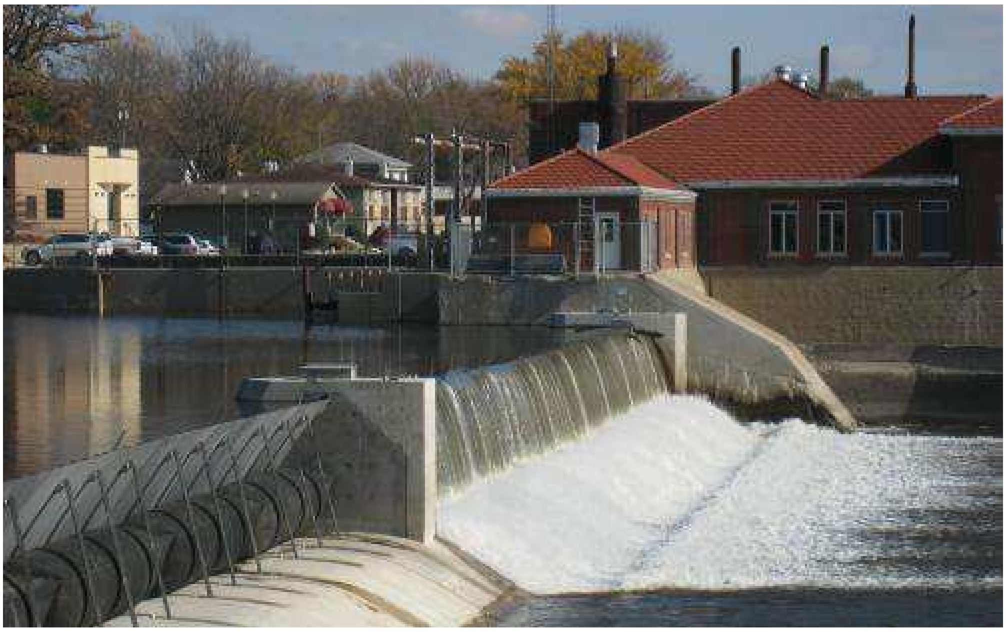


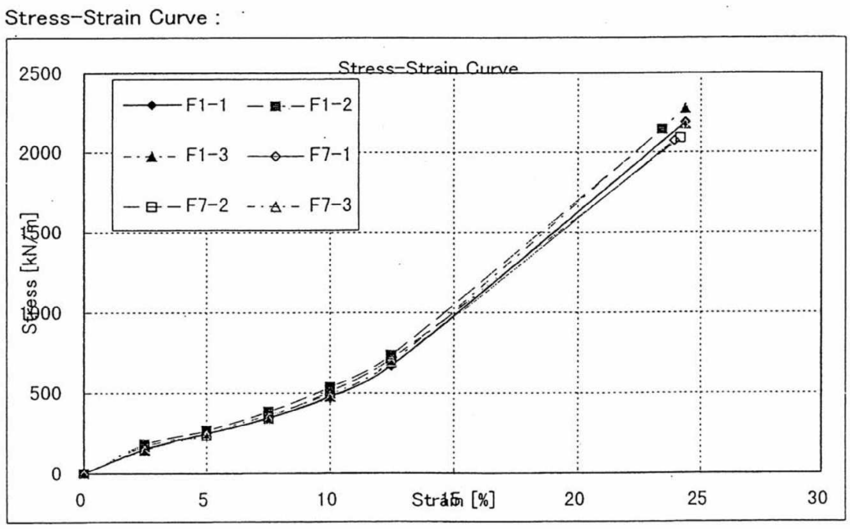
Figure 41 - Obermeyer weir (source: Obermeyer hydro inc.)

5.1.2 Properties of the membrane

The membrane sheet has a transmitting function, the membrane has to transfer the external hydraulic loads, the internal pressure and the self weight of the membrane to the foundation. The loads are transferred in transversal direction by normal forces in the membrane. The sheet material has to be strong enough to resist the static and dynamic loads on the construction. To resist the dynamic loads needs the sheet material a large fatigue strength. The sheet also has to be flexible otherwise can the sheet not be stored on in bottom of the construction.

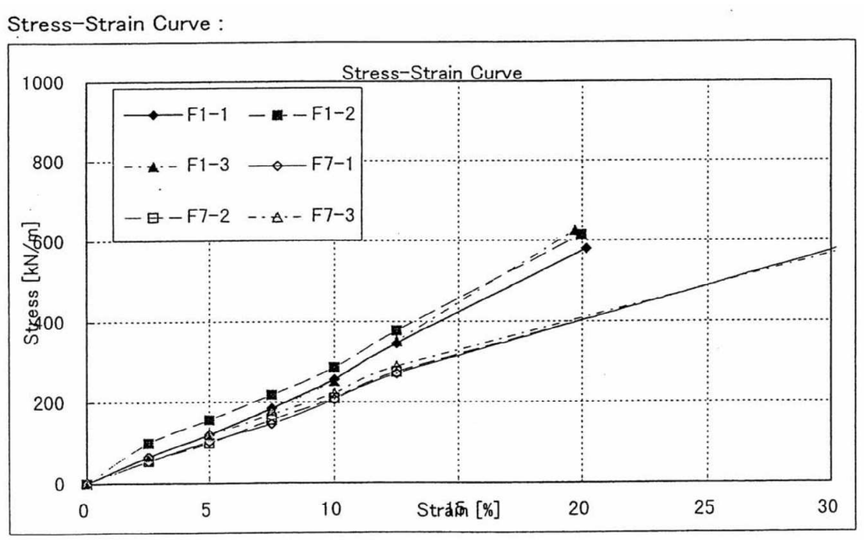
The sheet material is usually made from durable rubber types which are water and oil resistant. If the strength of the rubber is not sufficient the rubber could be reinforced with synthetic fibers like polyester or nylon. The reinforcement are woven mats that are placed in the rubber sheet during the manufacturing process. Because the load transferring is done in the transversal direction of the construction will this direction also contain the main reinforcement. The reinforcement in the longitudinal direction has to be less strong.
The reinforcement not only makes the sheet stronger but also stiffer. Due to the different amount of reinforcement in the transversal and longitudinal direction the material will become anisotropic. The anisotropic properties of the material are of importance for the degree of deformation during operation.

The strain stiffness of the reinforced sheets is not constant but changes with increasing loads. In Graph 2 is the transversal stress-strain relationship of the Ramspol barrier is represented. The graph represents tensile test for six different loads. It can be seen that the material becomes stiffer at higher loads because the strains are smaller for large loads. This property can be partly declared by an initial stretch of the reinforcement mats and partly by the non linear properties of the rubber. The strain of the membrane is for a small part dependant on the speed of loading, the moisture content of the material and the aging of the material. Because the crown height is a function of the perimeter has during design the effect of strain to be taken into account.



Graph 2 - Transversal stress-strain relationship of the rubber sheet of the Ramspol barrier (source: Rijkswaterstaat)

The strain stiffness in longitudinal direction is substantial smaller than in the transversal direction. The longitudinal stress-strain relationship for the Ramspolder barrier is represented in Graph 3. The tests are done for the same loads as the tests in the transversal direction.



Graph 3 - Longitudinal stress-strain relationship of the rubber sheet of the Ramspol barrier (source: Rijkswaterstaat)

5.1.3 Hydrostatic and hydrodynamic aspects

An inflatable construction differs from other types of constructions due its large deformation capacity and the way of transferring loads to the foundation. Not only the shape of the construction changes but also the stiffness changes due to varying internal and external loads. So the construction can take several shapes which makes the construction different to other types. The large formability of the construction is mainly reflecting in the dynamical behavior of the construction.

Important features of this type of construction are:

- There is a relation between the deformation of the construction and the state of the flow. When the water level on the upstream side becomes higher than the height of the inflated construction water will flow over crest of the construction. The water flow will follow the shape of the construction to under the downstream side. This causes a centrifugal force on the construction which causes a deformation of the construction. This relation causes flow induced vibrations in the construction.
- The deformation of the construction is under static loads relatively large compared to steel or concrete constructions. Due to this large deformation are also the response to dynamic loads relatively large.
- Due to vibrations of the inflatable construction the crest of the construction will also vibrate. This can results in a fluctuating discharge.
- The bottom of the downstream bottom has to be protected against the downward water jet over the construction.
- When a construction if partly filled with water a free water surface will arise in the construction. Due to movements of the construction the intern water could become in a sloshing state.

5.1.4 Filling of the construction

In the design of an inflatable construction is the type of filling an important aspect. Normally is the inflatable construction filled with water, air of a combination of both. By choosing the filling are the following aspects of importance:

- The time of filling and emptying of the construction.
- The sheet length in relation to the required crest height based on the external loads on the construction and required internal pressure.
- The loads on the membrane and the foundation.
- Should it be possible to change the pressure in construction to change the crest height? This is an important aspect for a water level regulating construction

- Is the construction placed in a region with strong and severe winters?

5.1.5 Design of the inflatable construction

Strict guidelines in designing inflatable constructions are hardly present. However the amount of sheet material becomes an important design aspect because the amount of sheet material has as strong influence on the total construction costs. A good measures for the amount and required type of membrane material is the product of required the membrane forces and the perimeter of the inflatable construction.

5.2 Functions of the flap gate

- Maintaining the upstream water level
- Retaining water
- Discharging water

5.3 Requirements to the flap gate

- The weir management according to Table 7 is maintained for the new situation
- The gates should be placed on the old Poirée construction
- The gate should be able to retain a upstream water level of 21.00 m +NAP.
- The width of the gate should be 60 m (with of the current Poirée part of the weir)
- The construction should be placed in the summer months.
- The gates should meet requirements according to the ARBO legislations
- The gate should be able to obtain a top discharge
- The gate should be able to obtain a more accurate discharge than the Stoney gates

5.4 Boundary condition

- The width of the gate should be 60 m.
- The bottom of the fixed construction is located at 15.95 m +NAP
- Water Levels upstream and downstream

5.5 First design choices for the inflatable flap gate

Before the first design if the inflatable flap gate is made, design choices that are important in calculation of the gate are determined. Choices with regard to the gate angel, the inflatable bladder, the filling of the bladder and the gate material are represented.

5.5.1 Gate angle

The angle of the flap has a certain influence on the pressure in the bladder. A angel between flap and weir bottom results in an internal pressure in the bladder . In closed condition, a straight flap gate usually makes an angle of 60° to 70° with the weir bottom (Boiten, 1995). In lowered position, the flap forms a continuous surface with the weir bottom. It is assumed to use an angle of 60° between bottom and gate. A gate angle of 60° and a upstream water level of 5.05 m represented in Table 12 results in minimum gate length of 5.8 m. An extra length is of 0.5 m is added. The total length, in cross sectional direction of the gate becomes 6.3 m. A cross section of the flap gate is presented in Figure 42.

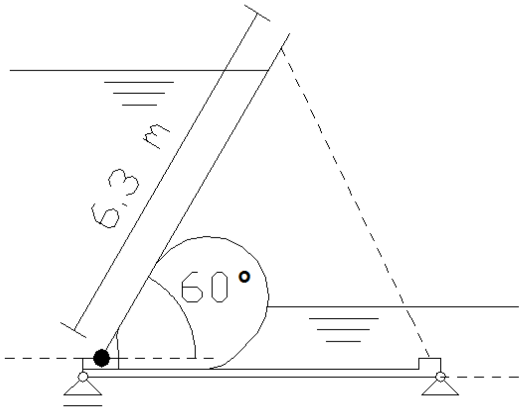


Figure 42 - Cross section of the flap gate

5.5.2 Inflatable bladder

In this section the number of bladders and the type of filling of the bladder are discussed.

5.5.2.1 *Number of bellows*

The bellow is located at the hinge between gate and bottom. Two types of bellow configuration are known namely:

- *One inflatable air bladder*

The usual and most commonly type that used is the left variant represented in Figure 43. The construction consist of a flap supported by one bladder. The pressure point between gate and bellow is located near the hinged connection between gate and bottom. The weight of the flap and the upstream water pressure result in an evenly distributed filling and emptying of the inflatable air bladder.

- *Two or more inflatable air bladders*

If high and thin flap gates are required, one can use a variant that consist of two inflatable air bladders placed upon each other. This variant is represented in the right figure of Figure 43. At this variant the pressure point is shifted from the hinged connection to the free top end.

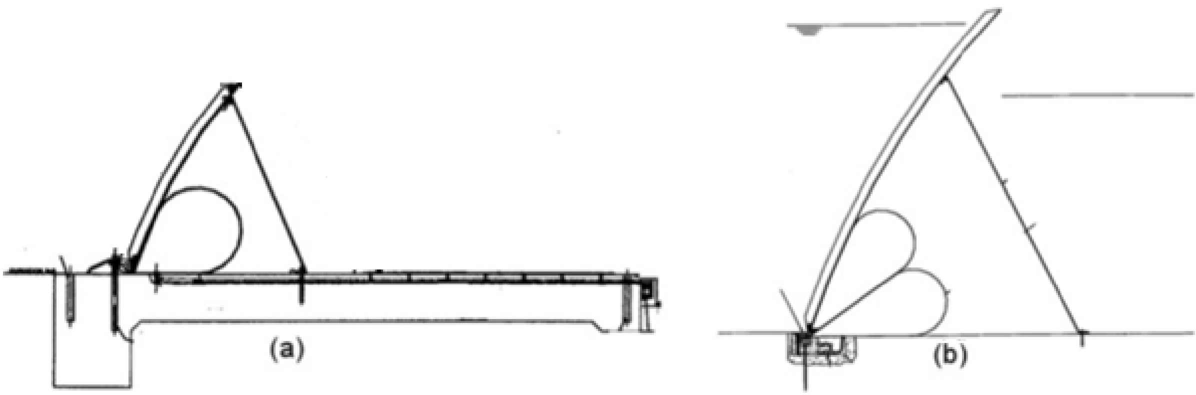


Figure 43 - Left: flap gate with a single bladder; Right: flap gate with multiple bladders upon each other (Source: Obermeyer Inc.)

Two inflatable bladders are advantageous for high and thin structures, the angel between bottom and gate is higher than for the single bladder type. This results in an shorter flap than for the single bladder variant. However the bladders are commonly the most expensive part of an inflatable construction. Two require more sheet material and more important, two air pumps are required to fill the bladders. Also the load distribution in a shorter flap becomes more intensive than for the single bladder type. It is chosen to design the weir with a single air bladder, however it has to be investigated if a thicker flap (made of concrete) is not too heavy for a single bladder.

5.5.2.2 Type of filling

The inflatable air bladders can be filled with air, water or a mixture of air and water. In most cases are the bladders filled with air. The main reason is that inflatable flap gates are often used in cold areas. In cold conditions it is advisable to use air filled bladders because water in the bladders could freeze. An option to prevent freezing is to heat the water before entering the bladders. Another option is to fill the bladders with nitrogen, this has a positive effect to aging of the rubber sheet. Nitrogen is a cheap filling because it can be extracted from the air. The advantages and disadvantages of the filings are represented in Table 15.

Table 15 - Advantages and Disadvantages of filling types

	Filling	
	Advantage	**Disadvantage**
Air filled	• Larger retaining heights can be obtained • A simple pump is required, a simple pump results in lower costs • It is relative easy to pump water in the inflatable dam	• Anchoring is necessary to prevent push up of the bladders • A air filled bladder is sensible to vibrations
Water filled	• Less vibrations due to sloshing • One pump required	• During cold conditions could the water in the bladders frees
Mixture filled	• When opening the weir only air has to be pumped into the bladders, due to the overpressure the water is squeezed out. • At closure of the weir only the water has to be pumped out of the bladders, the air is squeezed out the bladders by outside pressure (water pressure on the flap + weight of the flap)	• Possible sloshing of the water in the bladder • Two pumps are necessary, an air pump and an water pump • Investigation is necessary to obtain a suitable relationship between air and water

It is possible that parts of the Meuse will freeze. In 1940, 1956, 1963 parts of the Meuse were frozen. If the Meuse freeze also the water in the bladders will freeze. The result is an inoperable weir, the weir cannot be opened if the water in the bladders is not in fluid condition. It is advisable to use air instead of water as filling of the bladders.

5.5.3 Number of gate elements

The number of gate elements is important during construction of the gates. One large gate element over the total width of the Poirée part probably not the best option. It is more beneficial to divide the gates into a number of smaller gate elements. The width of the elements should be small enough to prevent large water level drops in canal section Linne if one of the gate section fails (e.g. a supporting bladder fail). Smaller elements are also beneficial for the purpose of transport. Transport of a 60 m wide gate efficient. Also the placement of a 60 m wide gate should be not the most sufficient option. It should be more sufficient to divide the gate into smaller gate elements

The width of the elements is calculated according to Equation 7. The discharge Q is chosen at 230 m^3/s, the average discharge of the Meuse. The result is a maximum with of 19.97 m witch result for a width of 60 m of the Poirée part in 3 elements. It has to be noticed that further investigation should be done to determine the width of the gate elements based on high and low discharges and the probability of failure of a gate element. More elements result in a higher accuracy of the discharge regulation.

5.5.4 Material: High strength concrete

High strength concrete a compressive strength class higher than C50/60. High-strength concrete is made by lowering the water-cement (W/C) ratio to 0.35 or lower.
Low W/C ratios and the use of silica fume make concrete mixes significantly less workable, which is particularly likely to be a problem in high-strength concrete applications where dense reinforcement cages are used. To compensate for the reduced workability, super plasticizers are commonly added to high-strength mixtures. High strength concrete is able to resist loads that cannot be resisted by normal-strength concrete. Not only can high strength concrete be used for more applications, it also increases the strength per unit cost, per unit weight and per unit volumes as well. The concrete mixtures have an increased modulus of elasticity which increases stability and reduces the deflections. High strength concrete has several advantages compared to normal strength concrete. Advantages are for example:

- Slender structures are possible
- Less material is needed and constructions become less heavy
- Larger spans are possible
- High durable constructions could be obtained which require less maintenance, especially in aggressive environments

Most of the disadvantages are due to a lack of adequate research under field conditions. However many of the issuers are currently being alleviated though the use of improved mixtures.
Disadvantages are for example:

- Increased quality control is needed in order to maintain the special desired properties.
- Careful material selection is necessary, high quality materials should be used.
- The minimum cover of reinforcement or minimum thickness of members may restrict the maximum benefits.
- Low water-concrete ratios require special curing requirements

Chosen is to design the gate in high strength concrete, this material requires less maintenance, is durable and is able to design in slender structures. The choice is not only made based on the material properties, advantages and disadvantages. In 2002 has M. van Helden (student of the Hogeschool van Utrecht) designed a inflatable flap gate for weir Sambeek. He investigated a steel gate and a plastic gate. The choice for steel or plastic results in a repetition of the report or van Helden. In addition, the design of high strength concrete weir gates is not yet applied in the Netherlands. However there are lock gates in high strength concrete applied for lock 124, located in Amsterdam. The gates are cheaper in terms of initial cost and maintenance, and are also more environmentally friendly. The gates are 6.5 m wide, 4.5 m high and have a thickness of 100 mm. The example of lock 124 indicates that hydraulic gates made of high strength concrete are possible.

(Pennsylvania State University) & (Cement & Beton Centrum)

5.6 Loads

Loads are acting on the gate. Three types of loads are distinguished namely permanent loads, variable loads and special loads.

- *Permanent loads*

Permanent loads includes loads that are relatively constant in time. In calculating a structure, a permanent load is assumed to remain unchanged in magnitude, line of action and point of application.

- *Variable loads*
  Variable loads are temporary loads that are not present during the total lifetime of the construction or loads which magnitude changes in time.
- *Special loads*

Partial load factors are safety factors. A load is multiplied with a load factor to obtain the characteristic value of the load. The characteristic load is used in various calculations to obtain a safe construction. In Table 15 the partial safety factors for loads are represented for the ultimate limit state. In Table 17 the safety factors for concrete and reinforcement steel are represented in the ultimate limit state according to Eurocode 2 (Comité Européen de Normalisation, 2011).

Table 16 - Partial factors for loads (source: Eurocode 2)

Design situations	Ψ
Permanent (Ψ_p)	1.2
Temporary (Ψ_t)	1.5

Table 17 - Partial load factors for materials for the ultimate limit state (source: Eurocode 2)

Design situations	γ_C (concrete)	γ_S (reinforcement steel)	γ_S (prestressing steel)
Permanent & temporary	1.5	1.15	1.15
Exceptionallly	1.2	1.0	1.0

5.6.1 Permanent Loads

In this section are only the permanent loads are represented that will be used in the calculations of the weir gate (Vrijling, et al., 2011).

5.6.1.1 *Dead weight*

The dead weight is the weight of the construction, in case of this report the concrete weir gate. The dead weight is dependent on the dimensions and the material of the gate. The dead weight is determined according to Equation 12.

$$S = V_{ol} g \rho_c$$

Equation 12 - Self weight

5.6.2 Variable loads

In this section only the variable loads are represented which will be used in the calculations of the weir gate (Vrijling, et al., 2011).

5.6.2.1 *Hydrostatic pressure*

The hydrostatic water pressure in any given point below water level is a function of the pressure head and the density of the water. In water at rest or in an uniform flow, the pressure head is equal to the water depth at a considered point. Largely varying flow velocities in open waterways corresponds to largely curved flow lines. In such cases the pressure head is not equal to the water depth. The equation for the hydrostatic pressure is:

$$p_w = \rho g h_i$$

Equation 13 - Water pressure

5.6.2.2 Waves

The most important waves are generated by wind. When no measurements are available, the significant wave height can be estimated according to the Bretschneider method. The formula for the significant wave height is represented in Equation 14.

$$H_s = 0.283 * \frac{u^2}{g} * \tanh\left(0.53 * \left(\frac{g * d}{u^2}\right)^{0.75}\right) * tanh\left(\frac{0.0125 * \left(\frac{g * F}{u^2}\right)^{0.42}}{\tanh\left(0.53 * \left(\frac{g * d}{u^2}\right)^{0.75}\right)}\right)$$

Equation 14 - Bretschneider formula to determine the significant wave height

Two parameter (fetch and wind velocity) should be known besides the already known water dept as represented in section 2.6 (depth = 4 m). The maximum fetch is determined by the measurement tool of Google Maps. The distance of the fetch is 2.35 km as is represented in the right map of Figure 44. For the maximum wind velocities the storm of 28 October 2013 is used (one of the heaviest storms measured in the Netherlands according to the KNMI). The maximum average of the wind velocity in the Netherlands is represented in the right figure of Figure 44. For the area of Linne is a maximum hour average wind velocity of 14 m/s and used according to Figure 44.

The effect of gust velocities is included to determine the peak wind velocities. The gust factor, defined as the ratio between a peak wind gust and mean wind speed over a period of time, can be used along with other statistics to examine the structure of the wind. Gust factors are heavily dependent on terrain conditions (roughness). The formula of the gust factor is presented in Equation 15. The factor f_T is dependent on the data presented in the right picture of Figure 44. The average velocities are presented in hour averages which corresponds with f_T = 1.1 (Benschop, 2005). The roughness length z_0 has for the area of Linne a value of 0.3 for a minimum of height of z = 5 according to Eurocode 1. A height of z = 1 m above surface is assumed. The result is a gust factor of <G> = 1.92 and a peak velocity of u = 1.91 x 14 = 26.74 m/s. According the Bretschneider method the wave height becomes 0.88m for peak velocities.

$$\langle G \rangle = f_T \times \left(1 + \frac{1.42 + 0.3\, ln\left(\frac{10^3}{u_{gust,x}\, t_{gust}} - 4\right)}{ln\left(\frac{z}{z_0}\right)}\right)$$

Equation 15 - Gust factor (Wieringa)

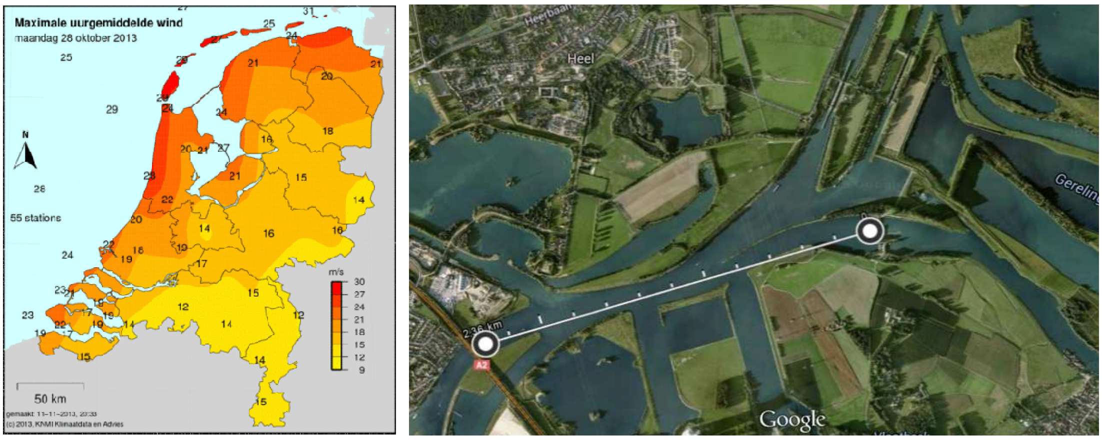


Figure 44 - Right: Maximum fetch for wave loads on the gate (Source: Google Maps); Left: Maximum hour average of the wind velocity (Source: KNMI)

There exist several methods to determine the loads on the gate due to waves. Five methods for calculating loads on wall due to non-breaking waves are represented in Table 18.

Table 18 - Methods to calculate the load on a wall due to non-breaking waves

Method	Design phase	Notes
Rule of thumb	Preliminary estimate	Conservative
Linear theory	Preliminary (and final) design	
Sainflou	Preliminary design	Simple method
Rundgren	Final design	
Goda	Final design	

Because one of the goals of this chapter is to check if it is possible to design a gate in high strength concrete, the rule of thumb is used. This a conservative method but also the most simple one. The formula for the rule of thumb is represented in Equation 16

$$F_{wave,max} = \frac{1}{2}\rho g H_s^2 + h_{up}\rho g H_s$$

Equation 16 - Rule of thumb to calculated a load on the gate due to waves

5.6.2.3 *Flow induced loads*

Two methods are possible to determine the load induced by a water flow against a wall. The most simple way is drawing up the balance impulse. The advantage of the balance impulse is simplicity of the method. The disadvantage of this method is that the total load on the wall is calculated, not the pressure distribution. The flow induced load is obtained from the balance area of Equation 17:

$$\sum \vec{F} + \rho * q * (u_{in} - u_{out}) = 0 \;\; or \; F_s = \rho * q * (u_{in} - u_{out})$$

Equation 17 - Flow induced load

The resulting force due to the water flow is the result of the difference between the flow in front of the weir and the outflow over the weir crest. The situation is represented in Figure 44. To obtain the flow velocities the rules for "Preservation of discharge" (volume and mass) and the "Preservation of energy" by Bernoulli are used.

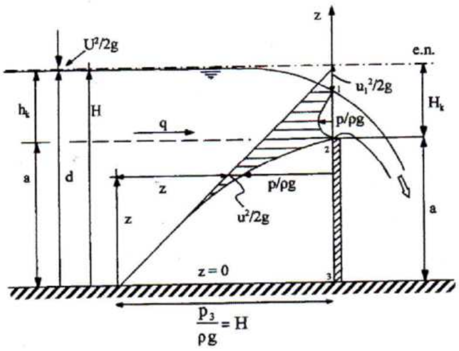


Figure 45 - Flow over a river weir (Source: TU delft, lecture notes CT3330)

Figure 45 represents the flow over a weir crest. It is represented that the energy heads at the upstream side and on top of the crest are equal which results in preservation of energy. It is assumed that the upstream water level is constant which results for a partly opened weir that the discharge on the upstream side is equal to the discharge on the crest of the weir. The preservation of volume is represented in Equation 18. The preservation of energy is represented in Equation 19.

$$q = u_{in}h_{up} = u_{out}h_{cr}$$

Equation 18 - Preservation of volume

$$H = h_{up} + \frac{u_{in}^2}{2g} = h_{cr} + \frac{u_{out}^2}{2g}$$

Equation 19 - Preservation of energy

The situation is easiest to solve if the discharge q is known. In this case the velocity u_{in} can simply be solved by Equation 18. The flow velocity u_{out} (on top of the crest) can be iteratively solved by Equation 20. Equation 20 is a substitution of Equation 19 into Equation 18.

$$u_{out} = \frac{q}{h_{cr}} = \frac{\frac{2}{3}h_{cr}\sqrt{\frac{2}{3}h_{cr}g}}{h_{up} - \frac{u_{out}^2}{2g}}$$

Equation 20 - Determination of u_{out}

The values of u_{in}, u_{out} and q are substituted in Equation 17 to determine the flow induced load. The flow induced load is only valid when water is flowing over the crest of the weir gate. However the exact distribution of the flow induces load on the gate is not exactly known, for reasons of simplification the distribution assumed to be linear distributed. This results in a distributed load over the weir gate as presented in Equation 21.

$$f_s = \frac{F_s}{L}$$

Equation 21 - Distributed flow induced load

5.6.2.4 Air pressure

The inflatable construction will be filled with air as was mentioned in section 5.5.4. The air filled bladder will push up the gate this results in load on the gate. The pressure in the bladder is evenly distributed, so everywhere in the bladder governs the same air pressure. The load resulting from the air filled bladder to the gate is described as represented in Equation 22.

$$P = p_a x_L$$

Equation 22 - Air pressure in the inflatable bladder

5.6.3 Special loads

5.6.3.1 Ice loads

Four ways of ice loads on hydraulic structures can be distinguished. The ways are:

- Terminal expansion
- Ice accumulation
- Collision
- Ice attachment

The two most important ways for the situation at Linne are ice accumulation and collision. As a result of low flow velocities, ice will accumulate to the structure. A flow beneath the ice causes a shear force along the ice which is in equilibrium with forces on the structure. Due to the accumulation of the ice in slowly flowing water, a static horizontal load is created on the structure.
The dynamic horizontal load on a structure is caused by colliding ice blocks. The blocks are transported by currents and wind. This is important for structures in rivers with considerable current and wind conditions. The impact of large ice blocks the collide against a structure can be compared with the impact of ships that run into a structure.

5.6.3.2 Drifting objects

Drifting objects could hit the weir gate, one should thing for example to drift wood. Drifting object are usually relatively light. However drifting objects could block the discharge opening of a weir. If lots of drifting material accumulates at the gates the same effect as accumulating ice could arise which induces a load on the gates.

5.6.3.3 Collision

The collision loads are the result of vessels that collide with the weir structure. However the navigational route is not in the vicinity of the weir, the navigational route and the inflow channel of to the weir is separated by small strip of land, as is presented in the right picture of Figure 44. Secondary it was mentioned in section 3.1.3 it is not allowed for vessels to navigate over a opened weir so vessels no vessels should approach the weir construction. Collision becomes for these reasons very unlikely.

5.7 Load distribution at the gate

In this section are the distribution of the moments and the shear forces in the gate (in cross sectional direction) determined. These knowledge is required to determine the (possible)reinforcement in the concrete gate. The moments and shear forces are represented as moment lines and shear force lines. Because the gate is rotating around one point during opening and closing do the moment and shear force distribution in the gate change with the angle between bottom and gate.

In Figure 46 represents the external loads on the construction for a closed gate, the external loads taken into account are loads due to the up (125 kN/m) and downstream water level (5.4 kN/m) and the wave loads (51 kN/m) on the upstream side of the gate. It has to be mentioned that the bottom of the current weir contains a height difference, as could be seen in Figure 28. On the upstream side of the gate the bottom is at 15.95 m +NAP and on the downstream side 15.30 m +NAP. The right picture of Figure 46 represents the support loads of the gate. The right figure represents that the gate construction actually consists of two constructions. The upper construction is formed by the gate and the second construction if formed by the bottom construction, including the bladder. The load distributions on the gate are determined in section 5.7.1 and section 5.7.2. Section 5.8 determined the reinforcement of the gate and design checks of the gate. Section 5.10 deals with the bottom construction of the weir.

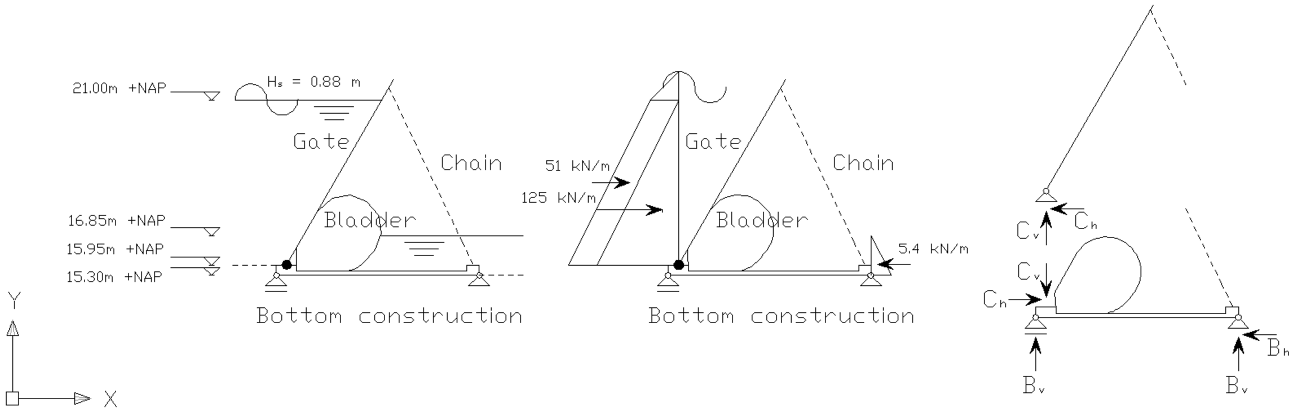


Figure 46 - Loads on the construction

As was represented in section 5.5.1 will the gate be placed under an angle of 60°. The design values for the upstream water level is 5.05 m as presented in Table 19. The gate is modeled as a straight plate with a cross sectional length of 6.3 m and a constant thickness of 0.20 m. The thickness is a assumed at 0.20 m but could be optimized to reduce the weight of the gate.

To determine the moment lines and shear force lines are first the used principles represented in 0, secondly are the load situations represented in 5.7.1 and at least are the moment and shear force lines represented in section 5.7.2. In Table 19 are the values given of the corresponding parameters that are used in the determination of the moment and shear force lines. In
Table 20 the loads on the gate are represented that are used to determine the moment and shear force lines.

Table 19 - Values to determine the moment and shear force lines

Description	Symbol	value	Comment
Density of water	ρ	1000 kg/m^3	
Density of concrete	ρ_c	2650 kg/m^3	
Gravitational constant	g	9.81 m/s^2	
Wave height due to wind waves	H_s	0.88 m	Represented in section 5.6.2.2.
Length of the gate	L	6.30 m	Represented in section 5.5.1
(assumed) Thickness of the gate	t	0.20 m	This assumption could be changed for the purpose of optimization of the gate design
Upstream water level	h_{up}	5.05 m	Represented in Table 12
Downstream water level	h_{down}	0.90 m	Water level canal section Roermond - level weir bottom (=16.85 m +NAP - 15.95 m +NAP)

Table 20 - Loads on the construction

Closed condition	
Loads	Type
$\boldsymbol{Q_1 := \frac{1}{2}\rho g h_{up}^2}$	Hydraulic load, dependant on the upstream water level
$\boldsymbol{Q_2 := \frac{1}{2}\rho g h_{down}^2}$	Hydraulic load, dependant on the downstream

	water level
$F_{wave,1} = \frac{1}{2}\rho g H_s^2 + h_{up}\rho g H_s$	Load due to non-breaking wind waves, dependant on: • wind velocity • depth of the river bottom
$F_s = \rho q(u_{in} - u_{out})$*	Horizontal force by the flow over the gate, dependant on: • crest height • discharge of the Meuse
$S = \rho_c gtL$	Self weight of the concrete gate for a thickness of 200 mm.
$S_{sub} = (\rho_c - \rho)gtL$	Load of the submerged gate for a gate thickness of 200 mm.

**As was concluded in section 5.6.2.3 is only de force on the wall determined but not the distribution over the wall of this load over the wall. To simplify the situation is assumed that the load is evenly distributed over the wall as is represented in section 5.6.2.3.*

One load on the gate is not presented in Table 20. According to Figure 43 and section 5.1.1.3 contains the gate a chain that prevents the gate for bouncing on the bladder. The determination of the magnitude and the force in the chains is made in section 5.7.1.1. The distribution of the moments and the distribution of the shear forces are of importance in determination of the reinforcement in the concrete gate. The determination of the moments and shear forced is done by use of the three conditions of equilibrium. The three equilibrium conditions are:

- *Equilibrium condition in x direction*
  The equilibrium condition in x direction implies that the sum of the horizontal forces is equal to zero as represented in Equation 23.

$$\sum F_x = 0$$

Equation 23 - Equilibrium condition in x direction

- *Equilibrium condition in y direction*
  The equilibrium condition in y direction implies that the sum of the vertical forces is equal to zero as represented in Equation 24.

$$\sum F_y = 0$$

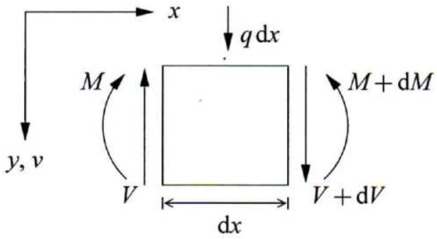
Equation 24 - Equilibrium condition in y direction

- *Moment equilibrium*
  The moment equilibrium condition implies that the sum of moments is equal to zero. A moment is a combination of a load and a distance Moments are usually defined with respect to a fixed reference point. The moment equilibrium is represented in Equation 25.

$$\sum T = 0$$

Equation 25 - Moment equilibrium

Moments are related to the shear forces, the relationships between moment and shear force is derived by expressing the equilibrium of a infinitesimal element dx of a beam loaded in bending. The equilibrium obtained by summing moments about an axis through the left hand side of the element of Equation 26.



Equation 26 - Element dx (Source: TU Delft, Lecture notes CIE 4190)

The result of the summing of moments is represented in Equation 27. This equation is valid in regions with distributed loads. The equation does not hold where there is a concentrated load on the construction excluding concentrated loads on the edges.

$$\frac{dM}{dx} = V$$

Equation 27 - Relation between shear forces and moments

As represented in Figure 48 to Figure 50 does the gate contain only one support, located at the bottom of the gate. Because the construction contains one support it becomes difficult to use framework programs like Matrixframe or to simplify the model to a boundary condition problem (becomes possible when the support becomes a fixed support).

The moment distribution and shear force distribution in the gate (in cross sectional direction) are determined by the internal load distribution. To determine the load distribution is the gate split up in cross sectional direction into two parts. The cut is made at the point when gate and bladder are do not connected anymore. The positive directions of moment and shear force at the location of the cut are represented in Figure 47.

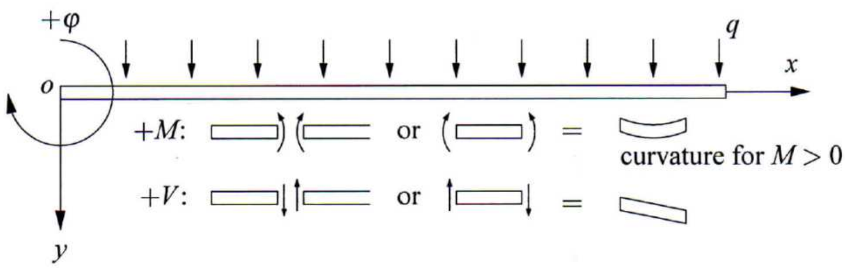


Figure 47 - Positive directions for moments and shear forces

The moments distributions on the left and right part of the cut are determined in M_{left} and M_{right} as is for example represented in
Equation 30. At the location of the cut holds that the moments and shear forces on the left and right side should be equal as represented in Equation 28.

$$M_{left} = M_{right}$$

$$\frac{dM_{left}}{dx} = \frac{dM_{right}}{dx}$$

Equation 28 - Moment and shear force at the location of the cut

5.7.1 Load situations

Before is started with the dimensioning are three load situations defined. The gate can be in closed, partly opened and in opened situation. In the situations another load distribution is valid. For the situations a load distribution have to be defined in order to obtain the governing load case. The load situations are defined in the sections 5.7.1.1 to 5.7.1.3. The load situations are corresponding to angle ranges between 0° and 60° cording to section 5.5.1, the ranges are:

- Closed situation $0° \leq \alpha < 53°$
- Partly opened situation $53° \leq \alpha < 0°$
- Opened situation $\alpha = 0°$

The closed situation corresponds with gate in raised position when no water is flowing over the crest of the gate. The partly opened situation represents a gate in raised position when water is flowing over the crest. In opened condition the gate is horizontally on the weir bottom.

5.7.1.1 Closed condition

In the closed condition the weir gates are in raised position such that no water flow over the gate is possible. The flap is raised by the inflatable bladder so the flap functions as a retaining wall for the water. The closed condition and the acting loads on the flap are represented in Figure 48. The loads that act on the structure in the closed condition are:

- Hydraulic loads (up and downstream)
- Wave loads

According to
Equation 30 and Equation 28 the two unknown variables p (air pressure in the bladder) and x (location of the cut) are able to be solved because there are two unknowns and two equations present.

The load of the chain is presented in Figure 48. The chain is in tension when the angle of the gate becomes larger than 60°, for smaller angles the tension in the chain is T = 0. Tension in the chain will be the result of:

- bouncing of gate on the bladder due to wave action.
- filling the bladder more than necessary which results in larger angles.

The tension load in the chain results in a kind of preload in the gate which prevents immediately bouncing of the gate for small wave actions. For a gate of 60° a tension of 5 kN/m is assumed in a chain to prevent bouncing of the gate for every wave load. It is assumed that the chain is connected 0.3 m below the top of the gate, for a more detailed model the most preferable location between gate and chain has to be determined. The tension in the chain and correspondingly the pretension in the gate can be obtained by:

- shortening of the chain length
- small overfilling of the bladder

It could be an option to increase or lower the chain tension and preload in the gate. This could be an option during storms which creates large waves.

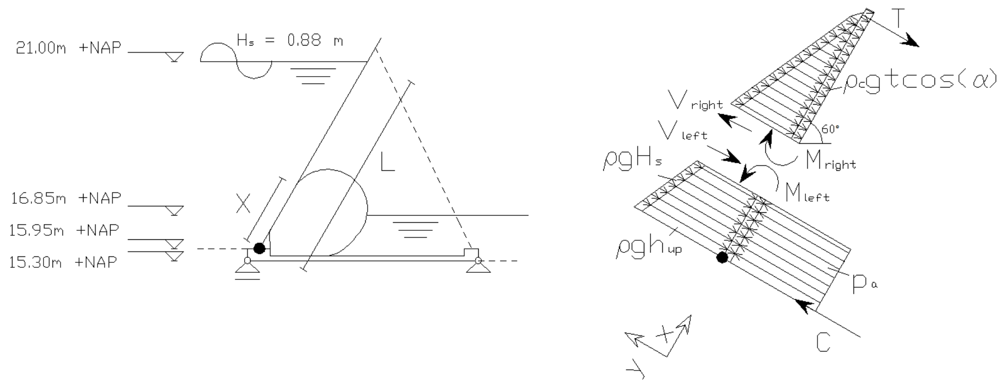


Figure 48 - Closed situation with acting loads

Figure 48 represents the loads on the weir gate. The x in the left picture represents the contact length between bladder and gate, L represents the length that upstream water level covers. The picture represents the acting loads on the gate. The magnitude of the loads change with the gate angel. The support load at point C is calculated according to Equation 24. The equation for the horizontal support reaction is represented in Equation 29. The support reaction is chosen perpendicular to the gate. The horizontal support reaction in Equation 29 is based on the right picture of Figure 48.

$$C + p_a x - \rho g h_{up} \frac{(L-x)}{L} x + \frac{1}{2} \rho g h_{up} \left(1 - \frac{(L-x)}{L}\right) x - \rho g H_s x - \rho_c g t \cos(\alpha)\, x - V_{right} = 0$$

Equation 29 - Horizontal equilibrium of the gate in closed condition

The values of the loads in Figure 48 are presented in Table 21. Table 21 presents the loads for a gate under an angle of 60°. The load P due to the supporting bladder is actually not known and should be calculated. The support load C is also not know because this load is dependent on the load P of the bladder. For the purpose to check if the horizontal equilibrium of Equation 29 is met, the calculated support load C and pressure load are also presented. It can be concluded that the horizontal equilibrium is met.

Table 21 - Loads on the gate in closed condition (60°)

Name	Expression	In closed position at 60° (ULS)
Wave load	$\rho g H_s x$	30 kN/m
Hydraulic loads	$\rho g h_{up} \frac{(L-x)}{L} x + \frac{1}{2} \rho g h_{up} \left(1 - \frac{(L-x)}{L}\right) x$	87 kN/m
Dead load of the gate	$x\, \rho_c g t \cos(\alpha)$	7 kN/m
Pressure force	$p_a x$	165 kN/m
Support load	C	100 kN/m
Shear force	V_{right}	133 kN/m

The moment distribution is determined according to the right picture of Figure 48. The figure represents the concrete gate including the acting loads. As represented in Figure 48 does the gate contain only one support, located at the bottom of the gate. Because the construction contains one support it becomes difficult to use framework programs like Matrixframe or to simplify the model to a boundary condition problem (becomes possible when the support becomes a fixed support). The moments left and right of the cut are represented in Equation 30.

$$M_{left} + \frac{1}{2}\rho g h_{up} \frac{(L-x)}{L} x^2 + \frac{1}{3}\rho g h_{up} \left(1 - \frac{(L-x)}{L}\right) x^2 + \frac{1}{2}\rho g H_s x^2 + \frac{1}{2}\rho_c g t x^2 \cos(\alpha) - \frac{1}{2} p_a x^2 + Cx = 0$$

$$M_{right} + \frac{1}{6}\rho g h_{up} \frac{(L-x)^3}{L} + \frac{1}{2}\rho g H_s^2 (L-x) + \frac{1}{2}\rho_c g t (6.3 - x)^2 \cos(\alpha) + T(6-x) = 0$$

Equation 30 - Moments of the left and right side of the cut for the closed condition

The effect is of the chain is represented in the moment lines of Figure 52 and the shear force lines of Figure 54. According to the moment lines the moment in the gate is 0 at and above the location of the chain which is correct according to the internal moment equilibrium.
The shear force line presents as small unevenness at the location of the chain. Above the location of the chain to the top of the gate has the shear force a value of 0 which corresponds with the internal shear force equilibrium.

5.7.1.2 Partly opend condition

The partly condition represents the load distribution during the opening and closing operation. In this situation is a certain amount of water discharged over the weir. The gate is rotating around the hinge. This results in water flow over the gate induced by a decreasing crest height. The situation and the acting loads are represented in Figure 49. The flow over the gate induces a load on the gate. The flow induced load was presented in section 5.6.2.3. The load is the result of the difference in flow velocity at location 1 and location 2 presented in Figure 49. The energy height remains constant from location 1 to location 2. The difference in flow velocity at location 1 and location 2 is the result of contraction of the flow lines, as presented in the figure.

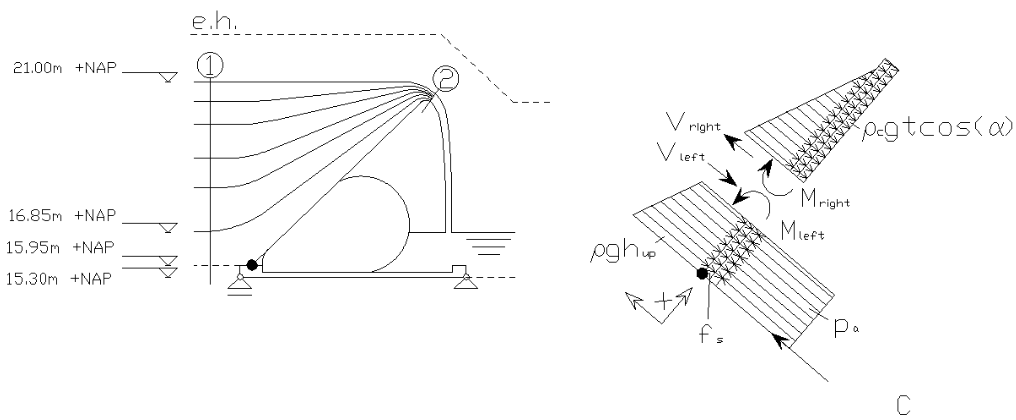


Figure 49 - Partly opened condition with acting loads

The support loads at point A are calculated according to Equation 23 and Equation 24. The equation for the horizontal support reaction is represented in Equation 31 and the equation for the vertical support reaction is represented in Figure 49.

$$C + p_a x - \rho g h_{up} \frac{(L-x)}{L} x + \frac{1}{2}\rho g h_{up} \left(1 - \frac{(L-x)}{L}\right) x - f_s x - 6.3\, \rho_c g t \cos(\alpha) - V_{right} = 0$$

Equation 31 - Horizontal equilibrium of the gate in partly opened condition

The values of the loads in Figure 49 are presented in Table 22. Table 22 presents the loads for a gate under an angle of 50°. The load P due to the supporting bladder is actually not known and should be calculated. The support load C is also not know because this load is dependent on the load P of the bladder. For the purpose to check if the horizontal equilibrium of Equation 31 is met, the calculated support load C and pressure load are also presented. It can be concluded that the horizontal equilibrium is met.

Table 22 - Loads on the gate in closed condition (50°)

Name	Expression	In closed position at 50° (ULS)
Flow induced load	$f_s x$	0 kN/m
Hydraulic loads	$\rho g h_{up} \frac{(L-x)}{L} x + \frac{1}{2} \rho g h_{up} \left(1 - \frac{(L-x)}{L}\right) x$	88 kN/m
Dead load of the gate	$x\ \rho_c g t \cos(\alpha)$	10 kN/m
Pressure force (P)	$p_a x$	149 kN/m
Support load	C	58 kN/m
Shear force	V_{right}	109 kN/m

The figure represents the concrete gate including the acting loads in the partly opened condition. As represented in Figure 49 does the gate contain only one support. The moments left and right of the cut are represented in Equation 32.

$$M_{left} + \frac{1}{2} \rho g h_{up} \frac{(L-x)}{L} x^2 + \frac{1}{3} \rho g h_{up} \left(1 - \frac{(L-x)}{L}\right) x^2 + \frac{1}{2} f_s x^2 \sin(\alpha) + \frac{1}{2} \rho_c g t x^2 \cos(\alpha) - \frac{1}{2} p x^2 - Cx = 0$$

$$M_{right} + \frac{1}{6} \rho g h_{up} \frac{(L-x)^3}{L} + \frac{1}{2} f_s x^2 \sin(\alpha) + \frac{1}{2} \rho g H_s^2 (L-x) = 0$$

Equation 32 - Moment determination in the plate for the partly opened condition

According to the two equations of Equation 32 and Equation 28 are the two unknown variables p (air pressure in the bladder) and x (location of the cut) be able to solve because there are two equations and two equations (Equation 28).

5.7.1.3 Opened condition

In opened conditions are the gates not damming the river, the gates are laying down on the weir bottom. Water can freely pass the weir construction. The situation and the corresponding loads are represented in Figure 50. The opened condition is represented as a construction on two supports. The area between the supports is used to store the inflatable bladders. The only acting load on the construction is the submerged self weight of the construction, the load is represented in Figure 50.

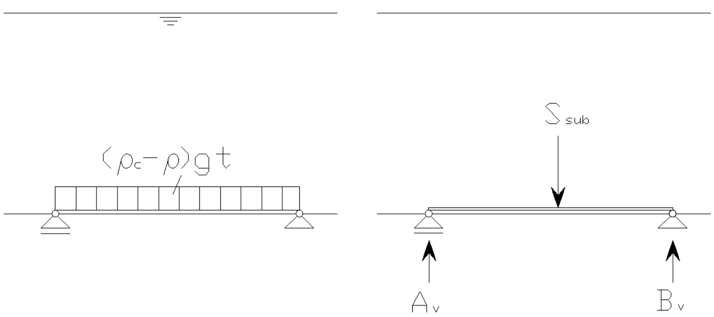


Figure 50 - Closed situation with acting loads

Compared to the closed and partly opened condition does the opened condition contain two supports. The situation contains two supports because the bladder is stored between the two supports. This condition does, compared to the other two situation only contain vertical loads and no horizontal loads. The support reactions A_v and B_v are represented in Equation 33.

$$A_v = B_v = \frac{1}{2} S_{sub}$$

Equation 33 - Support loads in opened condition

The moment distribution is determined according to Figure 51. The figure represents the concrete gate including the acting loads in the opened condition. As represented in Figure 50 does the gate contain two supports. The moments left and right of the cut are represented in Equation 34Equation 32.

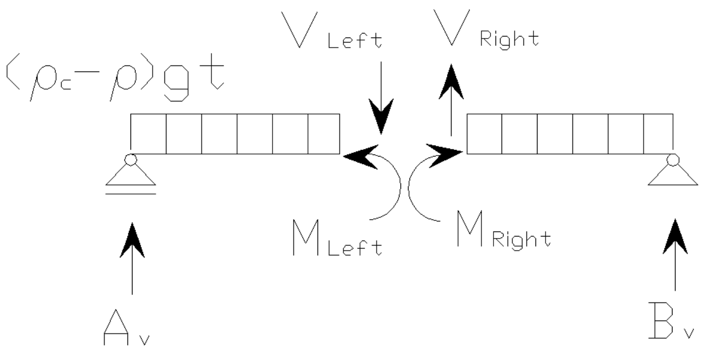


Figure 51 - Moment determination in the plate for the opened condition

$$M_{left} + \frac{1}{2}(\rho_c - \rho)gtx^2 - A_v x$$
$$M_{right} + \frac{1}{2}(\rho_c - \rho)gt(L-x)^2 - B_v(L-x)$$

Equation 34 - Moment determination in the plate for the opened condition

5.7.2 Results

In section 5.7.2.1 the resulting pressures in the bladder presented. The pressures are the result of solving the equations of the closed and partly opened condition. Also the resulting load of the bladder on the gates (which result from the equations $P = p_a x$) are presented. With the pressures in the bladder and the equations of the closed, partly opened condition the moment and shear force lines are constructed. The moment lines and shear force lines for the three different situation are plotted in Figure 52 and Figure 54 for angles between 60° and 0°.

5.7.2.1 Load of the bladder

By solving the equations for the closed and partly opened situation the air pressure in the bladder can be calculated. The air pressures in the bladder for different angles of the gate are presented in the middle column of

Table 23. It can be seen the pressures decrease substantially between 60° and 50°, from 50° the pressures remain relative constant. This could be declared by reshaping of the bladder. For lower angles the bladder becomes flatter, this is corresponds with lowering of the gate, as presented in the left picture of Figure 49. For larger angles the bladder becomes more in its final shape as is presented in the left picture of Figure 48. When the bladder becomes more to its final shape the pressure in the bladder increases however the shape of the bladder does not change any more.

Table 23 - Pressures in the bladder, contact length and loads of the bladder on the gate

Angle [°]	p_a [kN/m^2]	x [m]	P [kN/m]
60	51	3,25	165

55	43	3,81	164
50	36	4,13	149
45	29	4,42	130
40	29	4,49	130
35	29	4,49	129
30	29	4,49	129
25	28	4,49	128

5.7.2.2 Moment lines

The moment lines in represented in Figure 52 show the moment lines for different angles in ULS. The negative moments are larger than the positive moments, the large negative moments occur mainly in the closed situation so for the angle range 90° ≤ α < 53°. The positive moments occur only for the partly opened and opened situation. The largest moments are located on the downside part of the gate, so in the direction of the hinge. In Figure 52 are angles smaller than 25° not taken into account because the moments become not larger for smaller angles.

In Figure 53 is the enveloped area presented of the moment distribution represented for an angle range of 60° to 0°, the required range of the moments in the gate. The pink colored area represents the area of moments during opening or closing procedure. The maximum moments in for this range can be found for the gate angels of 60° for the negative moment and 0° for the positive moments. The moments corresponding to the angles are presented in Table 24. The moments are represented in the ultimate limit state (ULS) and in the serviceability limit state (SLS).

Table 24 - Maximum moments

Angle	Location on the x axis [m]	Moment [kNm/m]	
		SLS	ULS
60°	2.2	-93	-133
55°	2.5	-82	-116
50°	2.9	-33	-53
45°	3.7	-14	-25
40°	3.5	-14	-24
35°	3.6	-12	-20
25°	4.08	-13	15
0°	3.15	16	20

It has to be noticed that not all angles between 60° and 0° are represented in Figure 52. When all angles are presented will the enveloped moment lines of Figure 53 be more smooth compared to the presenting figure.

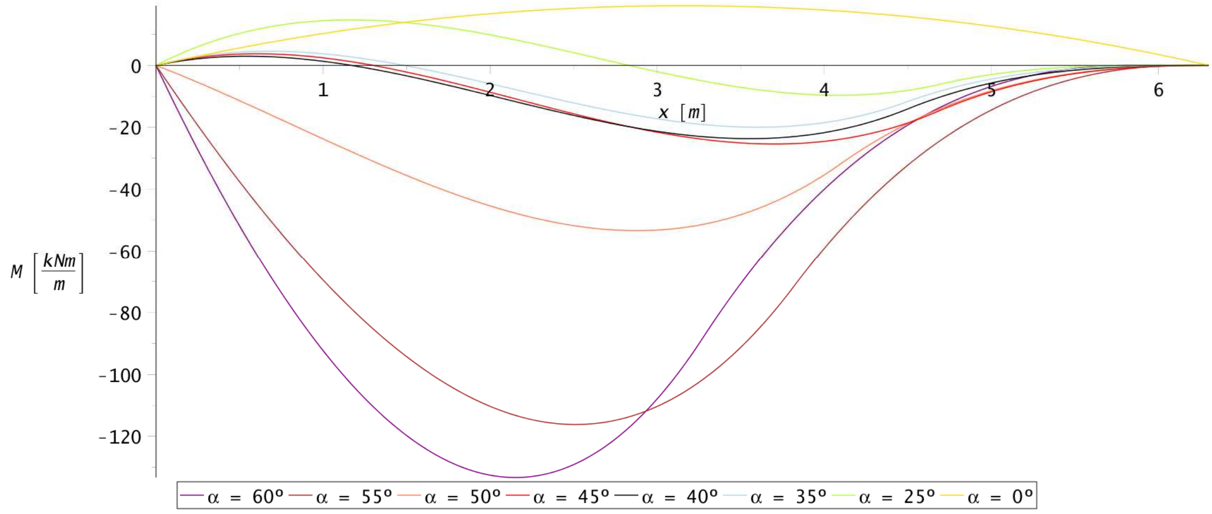


Figure 52 - Moment lines for the gate under different angles (ULS)

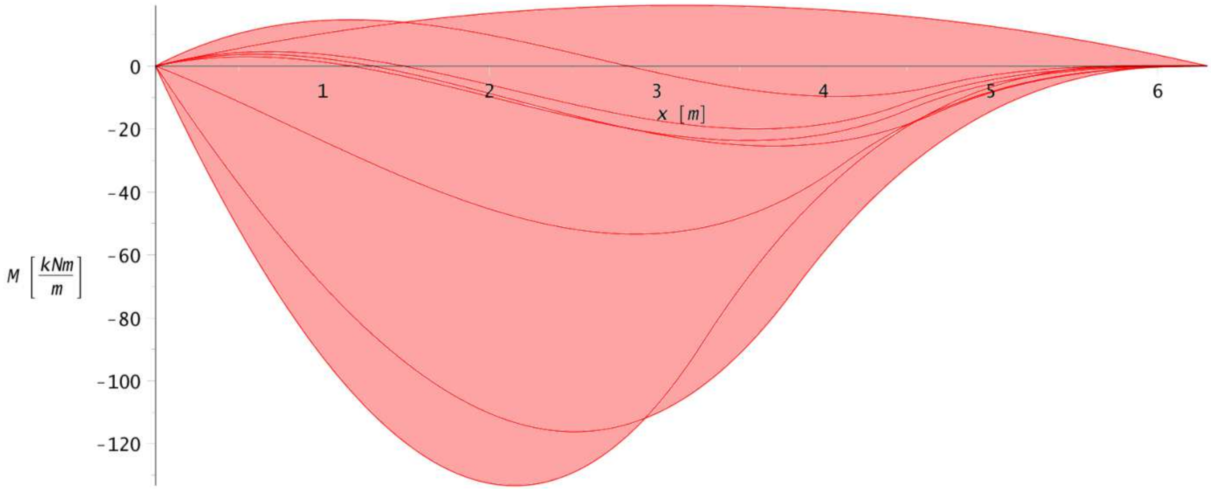


Figure 53 - Enveloped moment lines (ULS)

5.7.2.3 Shear force lines

The lines of the shear forces represented in Figure 54 show that the largest shear forces are located at the hinge and around the middle of the gate. The largest shear forces are according the figure corresponding with the large angels. For smaller angles are the shear forces relatively smaller. The smallest shear forces are located at the end of the gate and around 2 m above the hinge (= at x=0). As can be seen in the figure the shear forces in the direction of the free edge (at x = L) of the gate decrease but. The shear forces at the free edge is the result of the opened situation. The peaks in the shear lines represent the ending of the supporting bladder. It can be seen in Figure 54 the peaks move to the right for decreasing angles. This represents the flattening of the bladder due lower pressures in the bladder which occur due to lowering of the gate.

In Figure 55 is the enveloped area presented of the shear forces distribution presented for an angle range of 60° to 0°, the required range of the gate. The pink colored area represents the area of shear forces during opening or closing procedure. The maximum shear force in this range can be found for the gate angel of 60°. The largest shear force for this angle can be found at the hinge (at x = 0). The shear forces corresponding to the angles are presented in Table 25. The shear forces are represented in the ultimate limit state (ULS) and in the serviceability limit state (SLS).

Table 25 - Maximum shear forces

Angle	Location on the x axis [m]	Shear force [kN/m]	
		SLS	UGL
60°	0	-81	-114
55°	0	-58	-80
50°	4.1	24	38
45°	4.7	13	23
40°	4.5	13	22
35°	4.5	15	20
25°	0	13	28
0°	0	10	12

It has to be noticed that not all angles between 60° and 0° are represented in Figure 54. When all angles are presented will the enveloped moment lines of Figure 53 be more smooth compared to the presenting figure.

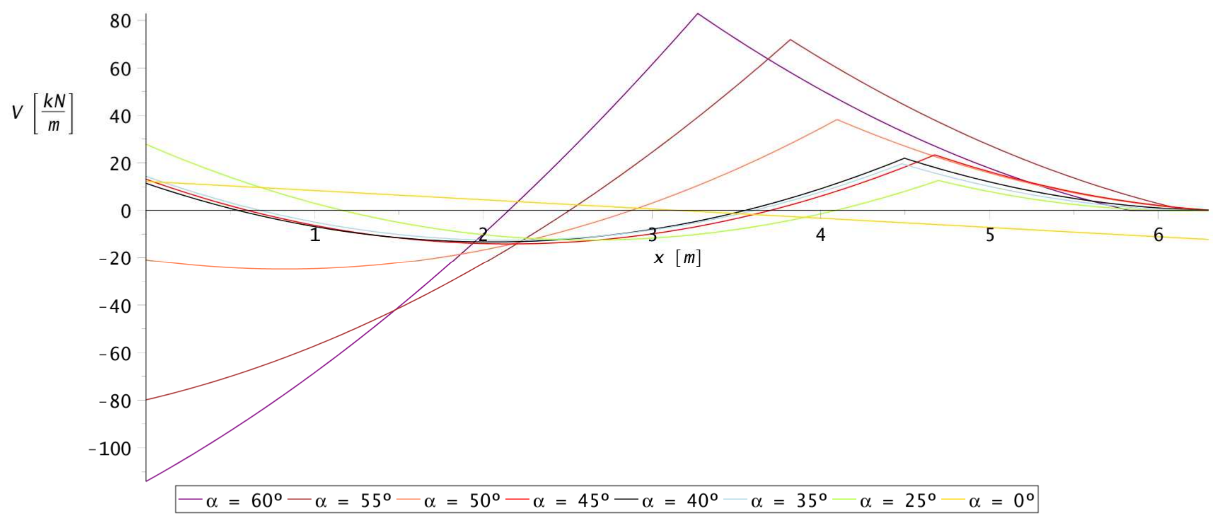


Figure 54 - Shear force lines for the gate under different angles

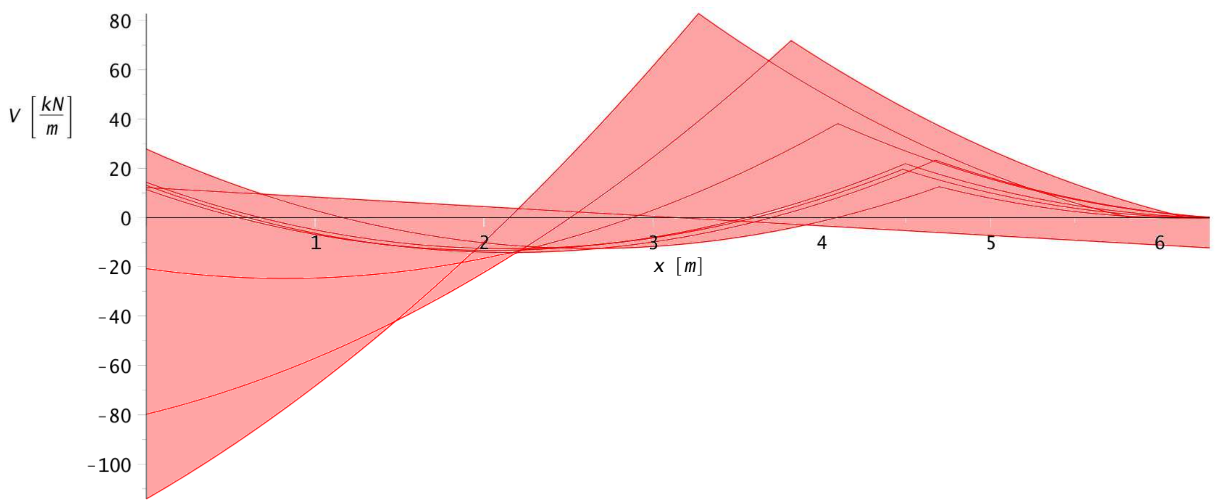


Figure 55 - Enveloped shear forces lines

5.8 Calculations for a concrete gate

The gate is determined in high strength concrete as was presented in section 5.5.4. It is assumed to use concrete class C90/105 because the durability of higher concrete classes are higher than lower classes. Possibly a gate construction for a smaller concrete classes is possible, however in this report is C90/105 assumed. As was presented in section 5.7 is the modeled as a straight plate with a cross sectional length of 6.3 m and a constant thickness of 0.20 m. An internal lever arm of 0.15 m is assumed in order to calculate the internal loads of the plate. In this section first the required reinforcement is determined, secondly the gate is checked to minimum and maximum reinforcement areas, cracks and failure of the gate.

5.8.1 Determination of the reinforcement

In this section the required reinforcement in the plate is determined. First the determination of the bending reinforcement is made and hereafter the determination of the shear force reinforcement. The properties of concrete (C90/105) and steel (B500) that are used in the calculations are represented in Table 26.

Table 26 - Design values for concrete (C90/105) and steel (B500)

Symbol	Value	description
d	150 mm	Internal lever arm
h	200 mm	height of the gate
f_{ck}	90 N/mm^2	Characteristic compression strength of concrete
f_{ctm}	5.0 N/mm^2	Characteristic tensile strength of concrete
E_{cm}	44000 N/mm^2	Young's modulus of concrete
ε_{cu3}	2.6 ‰	Strain
γ_c	1.15	Safety factor for steel
f_{yk}	500 N/mm^2	Characteristic strength of steel (B500)
E_s	210000 N/mm^2	Young's modulus of steel
γ_s	1.15	Safety factor for steel

5.8.1.1 *Determination of the bending reinforcement*

Bending and the combination of bending and normal forces are frequently occurring in concrete constructions. The determination of longitudinal reinforcement is based on the horizontal equilibrium according to Equation 23 and the moment equilibrium according to Equation 25. The internal loads in a cross are presented in the right picture of Figure 56. The used equations for N_s and N_c are presented in the Equation 35 to Equation 37.

$$N_s = A_s \frac{f_{yk}}{\gamma_s}$$

Equation 35 - Force due to reinforcing steel

$$N_c = \alpha b x_u \frac{f_{ck}}{\gamma_c}$$

Equation 36 - Force due to compression strength of the concrete

$$N_s = N_c = \frac{M_{Ed}}{d - \beta x_u} \approx \frac{M_{Ed}}{0.9d}$$

Equation 37 - Moment in relation to normal forces in the construction d = internal lever arm (assumed at 150 mm)

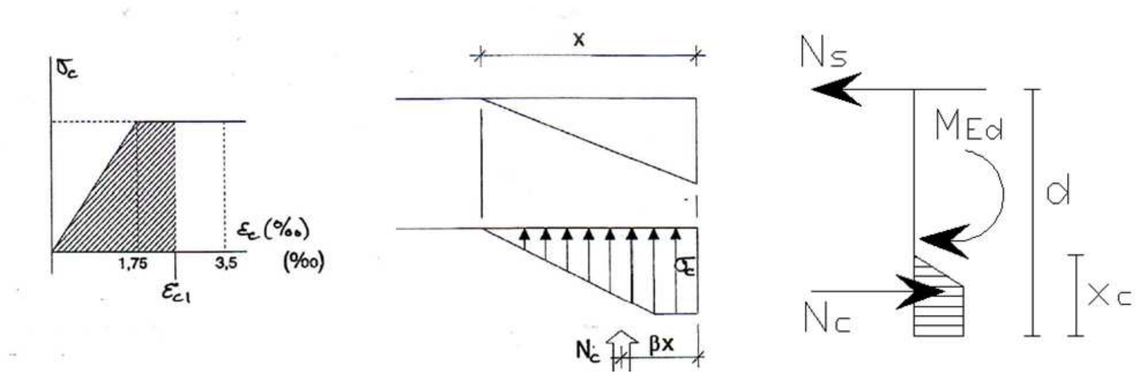


Figure 56 - Illustration of the compression zone (source : Tu Delft, lecture notes CT2052)

The factor α in Equation 36 determines the fullness of the stress distribution in the concrete based on the maximum strain, this factor is in Dutch called the "volheidsfactor". The factor β in Equation 37 determines the location of the resulting compression forces of the concrete, this factor is in Dutch called the "afstandsfactor". The factor is represented in the middle picture of Figure 56. Until concrete class C55/67 is usually used α_c = 0.75 and β = 0.39 for the frequently occurring situation $\varepsilon_{c1} = \varepsilon_{cu3}$. However for the weir gate is designed for concrete class C90/105. The factors α and β should be recalculated.

The factor α is calculated by determining the shaded area in the left picture of Figure 56 and dividing it to ε_{c1}, the compressive strain in the concrete at the peak stress f_c. However for case of simplicity are the factors not calculated in this report but just represented. For concrete class C90/105 holds α_c = 0.56 and β = 0.34 (Braam, 2009).

The results for the largest negative and larges positive moments found in section 5.7.2.2 are represented according to Equation 35 to Equation 37 in Table 27.

Table 27 - Results for the larges positive and negative moments

Symbol	Value	Notes
Largest negative moment of M_{Ed} = -133 kNm/m		
N_c	985 kN/m	
N_s	985 kN/m	
A_s	2266mm^2/m	
x_u	29 mm	Calculated for a gate section with a width of 1 m*
Largest positive moment of M_{Ed} = 20 kNm/m		
N_c	148 kN/m	
N_s	148 kN/m	
A_s	340 mm^2/m	
x_u	5 mm	

**The moments lines according to Figure 52 is constant in longitudinal direction of the gate. This means that on every location in longitudinal direction the moment lines as presented in Figure 52 are valid.*

For the reinforcement for the largest negative moment (M_{Ed} = -133 kNm/m) reinforcement bars with a diameter of 16 mm are assumed. A diameter of 16 mm results in12 bars per meter with a center-to-center distance of 83 mm. For the reinforcement for the largest positive moment (M_{Ed} = 20 kNm/m) reinforcement bars with a diameter of 8 mm are assumed. A diameter of 8 mm results in 7 bars per meter with a center-to-center distance of 142 mm.

5.8.1.2 Shear force reinforcement

In the plate are not only moments present but also shear forces. The shear load is a force that tends to produce a sliding failure in the concrete gate along a plane that is parallel to the direction of the force. It is not common (and advisable) to use shear force reinforcement in plates because this is a very laborious and time consuming activity (Eurocode 2). To reduce costs is it advisable to increase the height of the plate instead of use shear force reinforcement if the shear resistance of the plate is not sufficient. The minimum shear resistance is determined according to Equation 38 with use of Equation 39. In this calculation no axial load due to prestressing is present in the cross section, this results in $\sigma_{cp} = 0$ ($=N_{Ed}/A_c$).

$$V_{Rd,c} = \left(v_{min} + \left(k_1 \sigma_{cp}\right)\right) bd$$

Equation 38 - Minimum shear resistance

$$v_{min} = 0.035 k^{3/2} \sqrt{f_{ck}}$$

$$k = 1 + \sqrt{\frac{200}{d}} \leq 2.0$$

Equation 39 - Minimum shear stress

The results for the largest negative and larges positive moments found in section 5.7.2.3 are represented according to Equation 38 and Equation 39 in Table 27.

Table 28 - Results for the larges shear force in the construction

Symbol	Value	Notes
	Largest shear force of V_{Ed} = 114 kN/m	
v_{min}	1 N/mm^2	
$V_{Rd,c}$	141 kN/m	

The minimum shear resistance of the concrete gate is smaller than the largest shear force in the construction according to Equation 40 . The result is that no reinforcement is required to resist the shear forces.

$$V_{Rd,c} \geq V_{Ed}$$

Equation 40 - Design check to determine if shear reinforcement is required

5.8.2 Checks

In this chapter are several check represented to determine if the construction fulfils the requirements as presented in Eurocode 2. The checks that are taken into account are:

- Height of the concrete gate (section 5.8.2.1).
- Reinforcement check for the minimum reinforcement area (section 5.8.2.2)
- Reinforcement check for the maximum reinforcement area (section 5.8.2.3)
- Check of the cracking moment (section 5.8.2.4)
- Check of the moment of failure (section 5.8.2.5)

5.8.2.1 Height of the concrete gate

For the determination of the moment and shear force lines is a plate thickness of 200 mm assumed. The thickness of the plate is determined according to Equation 41.

$$h = d + \frac{1}{2}\emptyset + c_{nom}$$

Equation 41 - Height of the concrete plate

The concrete cover is required to protect the reinforcement in the gate. The concrete cover is the distance between the surface of the reinforcement closest to the nearest concrete surface (including stirrups and surface reinforcement) and the nearest concrete surface.

The nominal cover shall be specified on the drawings. It is defined as a minimum cover, c_{min}, plus an allowance in design for deviation Δc_{def}. The nominal cover is represented in Equation 42 and the minimum cover in Equation 43.

$$c_{nom} = c_{min} + \Delta c_{dev}$$

Equation 42 - Nominal concrete cover

$$c_{min} = \max\left(c_{min,b};\ c_{min,dur} + \Delta c_{dur,\gamma} - \Delta c_{dur,st} - \Delta c_{dur,add}; 10\ mm\right)$$

Equation 43 - Minimum concrete cover

The thickness of the components presented in Equation 43 can be found in section 4.4.1.2 of EC2. The value of the factor $c_{min,dur}$ is determined by the durability of the cover. The thickness of $c_{min,dur}$ is based on two conditions namely:

- *Environmental conditions*
  Environmental conditions are chemical and physical conditions to which the structure is exposed in addition to the mechanical actions. Parts of the gate are constantly in contact with water, this corresponds with environmental class XC2. More information of this environmental class can be found in table 4.1 of the EC2 (Eurocode2).
- *Structural classification*
  The recommended Structural Class (service life of 50 years) is 4 for the indicative concrete strengths below C35/45. For higher concrete classes is the structural class reduced with one class to class 3.

The thickness of the components presented in Equation 43 are represented in Table 29. This result is a minimum concrete cover of 20 mm.

Table 29 - Components to determine the minimum concrete cover

Symbol	Thickness [mm]	Description
$c_{min,b}$	20	Table 4.1 of EC2
$c_{min,dur}$	20	Table 4.4 of EC2
$\Delta c_{dur,\gamma}$	0	
$\Delta c_{dur,st}$	0	
$\Delta c_{dur,add}$	0	

By use of Equation 41 and the values presented in Table 29 and section 5.8.1.1 is found that the thickness of the plate should be 178 mm. This is smaller than the assumed 200 mm chosen for the model. In optimization of the design should this be an aspect which should be investigated. However will this thickness of 200 mm be used in this chapter because the function of chapter 5 is to investigate if it is possible to design a concrete weir gate.

5.8.2.2 Reinforcement check for the minimum reinforcement area

In section 9.2.1.1 of Eurocode 2 is the requirement presented for the minimum reinforcement. The longitudinal tension reinforcement should not be taken less than $A_{s,min}$. A minimum reinforcement area is determined to prevent brittle failure of a construction. The determination of the minimum reinforcement area is represented in Equation 44.

$$A_{s,min} = 0.26\frac{f_{ctm}}{f_{yk}}bd$$

Equation 44 - Minimum reinforcement area

The result is a minimum reinforcement area is $A_{s,min}$ = 390 mm^2/m. The minimum reinforcement area is smaller than the reinforcement are determined in section 5.8.1.1. This holds for the largest negative moment as well for the maximum positive moment. The check for the minimum area of reinforcement is presented in Equation 45.

$$A_{s,min} \leq A_s$$

Equation 45 - Check for the minimum reinforcement are

5.8.2.3 Reinforcement check for the maximum reinforcement area

The maximum reinforcement is determined by the influence of any redistribution of the moments on all aspects of the design. The redistribution of bending moments may be carried out with explicit check on the rotation capacity. For concrete classes higher than C50/60 is the rotation capacity presented in Equation 46 (equation 5.10 (b) of EC2) and Equation 47 (from national annex section 5.5). A maximum reinforcement area is just as the minimum reinforcement area determined to prevent brittle failure of a construction.

$$\delta \geq k_3 + k_4\frac{x_u}{d}$$

Equation 46 - Rotation capacity for concrete classes higher than C50/60

$$k_3 = \frac{7f}{\varepsilon_{cu3}10^6 + 7f}$$

$$k_4 = 1.0$$

Equation 47 - Factors for concrete classes higher than C50/60

The tension f in the expression of k_3 is represented by Equation 48. Because no pretention is present the tension f reduced to the design tension of the reinforcement steel.

$$f = \frac{\left(\frac{f_{pk}}{\gamma_s} - \sigma_{pm\infty}\right)A_p + f_{yd}A_s}{A_p + A_s} = f_{yd}$$

Equation 48 - Tension

Because the rotation capacity is not defined with confidence is advised in Eurocode 2 to take the distribution of moments not into account. This results in a rotational capacity of δ = 1.

Table 30 - Values according to EC2 for determination of the maximum reinforcement

Symbol	Value
f	435 N/mm^2
δ	1
k_3	0.54
k_4	1.0

From Equation 46 the maximum value of the height of the compression zone x_u is determined. By means of the obtained (maximum) value of x_u and the use of Equation 35 and Equation 36 is the reinforcement area A_s determined. This reinforcement are determined from the rotational capacity of the construction is referred as the maximum reinforcement area. The result of the described procedure is a maximum height of the compression zone of $x_{u,max}$ = 69 mm and a maximum reinforcement area of $A_{x,max}$ = 5340 mm^2/m. It can be concluded that according to the check represented in Equation 49 and the reinforcement area as presented in Table 27 the requirement with respect to the maximum reinforcement is met.

$$A_{s,min} \leq A_s$$

Equation 49 - Check for the maximum reinforcement area

At last is checked if the compression zone x_u is not to large. The requirement for the maximum height of the compression zone can be found in the national annex of EC2. The requirement is presented in Equation 50. The expression is actually the same expression as the combination of Equation 46 and Equation 47 for the given values of k_3, k_4, f and δ. The resulting maximum height of the compression zone $x_{u,max}$ becomes for that reason $x_{u,max}$ = 69 mm.

$$x_u \leq \frac{\varepsilon_{cu3}\, 10^6\, d}{\varepsilon_{cu3}\, 10^6 + 7f}$$

Equation 50 - Maximum height of the compression zone

It can be concluded that according to the check represented in Equation 50 and the compression zone height as presented in Table 27 the requirement with respect to the maximum height of the compression zone is met.

$$x_{u,max} \geq x_u$$

Equation 51 - Check for the maximum height of the compression zone

5.8.2.4 Cracking moment (BGT)

As already remembered in section 5.8.2.2 and 0 is brittle failure of the gate not acceptable. It is required that in every situation the moment of failure for a bending induced reinforced construction should be smaller than the cracking moment. The cracking moment for a unreinforced cross section is represented in Equation 52. The cracking moment is determined in BGT because the construction does not fail at the cracking moment. A cracking moment of a reinforced cross section is a bit higher because the steel is under tension and consequently contributes to the load transfer. However this influence is relatively small so neglected in the equation.

$$M_{cr} = \frac{1}{6} b h^2 f_{ctm,fl}$$

Equation 52 - Cracking moment

$$f_{ctm,fl} = (1.6 - h) f_{ctm}$$

Equation 53 - Mean value of axial tensile strength of concrete

The result of Equation 52 is a cracking moment of 47 kNm/m, which is smaller than the maximum negative moment. The result is the occurrence of cracks in the plate. According to Eurocode 2 is the maximum crack width for a construction in exposure class XC2 is w_{max} = 0.3 mm. The crack width of the construction is calculated for a gate segment of 1 m wide and the moment in SLS, a moment of 93 kNm/m. Because the calculation of the crack width is done in SLS the height of the compression zone for SLS has to be calculated. In Figure 57 is the compression zone of a plate in SLS represented.

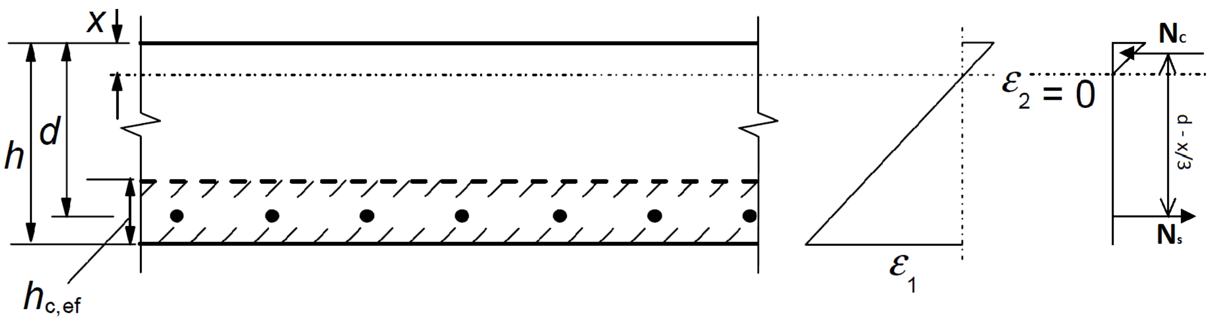


Figure 57 - Compression zone in SLS (source: Eurocode 2)

The height of the compression zone in SLS is calculated according to the moment equilibrium around the neutral axis. The calculation is done for a strip of 1 m wide and represented in Equation 54. This results in a height of the compression zone of 47 mm.

$$\alpha_e = \frac{E_s}{E_{cm}} = \frac{210000}{44000} = 4.77$$

$$b\ \frac{x^2}{2} = \alpha_e A_s\,(d - x) \quad \rightarrow \quad 1000\frac{x^2}{2} = 4.77 * 2266 * (150 - x)$$

Equation 54 - Moment equilibrium

The crack width is represented according to Equation 55.

$$w_k = s_{r,max}\,(\varepsilon_{sm} - \varepsilon_{cm})$$

Equation 55 - Crack width

The expression of ε_{sm} - ε_{sm} according to EC2 is presented in Equation 56. The factor k_t depends on the duration of the load because the load duration is of a long term holds according to EC2 k_t = 0.4.

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_s - k_t \dfrac{f_{ctm}}{\rho_{p,eff}}\left(1 + \alpha_e \rho_{p,eff}\right)}{E_s}$$

Equation 56 - Difference between the mean strain under loads and the mean strain between cracks

The expression for the maximum crack spacing ($s_{r,max}$) according to EC2 is presented in Equation 57. The factor c represents the thickness of the concrete cover as presented in section 5.8.2.1 (c = 20 mm). The factor k_1 is a coefficient which takes account of the bond properties of the bonded reinforcement, for high bond bars is k_1 = 0.8. The factor k_2 is a coefficient which takes account of the distribution of strain, for bending holds k_2 = 0.5. According to section 5.8.1.1 is the assumed bar diameter to resist the largest bending moments 16 mm.

$$s_{r,max} = 3.4c + \frac{0.425\, k_1 k_2 \phi}{\rho_{p,eff}}$$

Equation 57 - Maximum crack spacing

The expression for the steel stress is presented in Equation 58. The moment in the expression is the moment in SLS instead of ULS.

$$\sigma_s = \frac{M_{Ed}}{\left(d - \frac{x}{3}\right) A_s} = \frac{93 * 10^6}{\left(150 - \frac{47}{3}\right) * 2266} = 305\ {}^{N}/_{mm^2}$$

Equation 58 - Steel stress

The factor $\rho_{p,eff}$ represents the effective reinforcing ration, the ration is dependent on the reinforcement depth $h_{c,ef}$ as presented in Figure 57. The expression of the effective reinforcing ratio is presented in Equation 59. The value of $h_{c,ef}$ is the lesser of 2.5(h-d), (h-x)/3 or h/2. For this situation the value of $h_{c,ef}$ is $h_{c,ef}$ = 51 mm. The width of the effective tension area $A_{c,eff}$ becomes for a gate strip of 1 m wide $A_{c,eff}$ = 51000 mm^2. The value of the reinforcement area is presented in Table 27 and multiplied with 1 m.

$$\rho_{p,eff} = \frac{A_s}{A_{c,eff}} = \frac{2266}{51000} = 0.044$$

Equation 59 - Effective reinforcement ratio

The crack width becomes, according to Equation 55 the crack width of the gate is 0.12 mm. This is smaller than the maximum crack width of w_{max} = 0.3 as was mentioned in the beginning of this section. This means that the crack width is small enough with respect to the maximum crack width according to EC2. The results of the check of the crack width are presented in Table 31.

Table 31 - Results of the crack width analysis

Symbol	Value
σ_s	305 N/mm^2
$\rho_{p,eff}$	4.4%
α_e	4.77
x	47 mm
$\varepsilon_{sm} - \varepsilon_{cm}$	0.0012 ‰
$s_{r,max}$	98.6 mm
w_k	0.12 mm

As can be concluded from

Table 31 is the calculated maximum crack width smaller than the maximum crack with according to Eurocode2. This means that the requirement with respect to the maximum crack with is met. It has to be mentioned that in this section only cracking due to moments are taken into account. In practice also cracking due to torsion should taken into account.

5.8.2.5 Moment of failure (UGT)

The moment of failure for which the gate will fail. The moment of failure is determined according to Equation 60. The compression strength of concrete is represented in Equation 36. It has to be mentioned that for the height of the compression zone is used $x_{u,max}$ as presented in section 5.8.2.3 because it is the compression zone height according to the maximum reinforcement area.

$$M_u = N_c(d - \beta x_{u,max})$$

Equation 60 - Moment of failure

The calculated moment of failure is set at M_u = 294 kN/m. The moment of failure is smaller than the largest negative moment presented in section 5.7.2.2. The construction meet the requirement for the moment of failure as presented in Equation 61.

$$M_u \geq M_{Ed}$$

Equation 61 - Design check for the moment of failure

5.8.3 Reinforcement detail

In this section is the reinforcement determination of section 5.8.1.1 transferred to a reinforcement plan. The results of section 5.8.1.1 are represented in Table 32. The reinforcement as presented in the table is calculated in the y-direction as presented in Figure 58. The moment distribution as presented in Figure 52 are identically in the longitudinal direction (x-direction in the figure). At the top of the gate the diameter of the longitudinal reinforcement bars could be decreased. However in situations with reinforcement two directions (e.g. in plates) often reinforcement webs are used. For reinforcement of the positive moment of 20 kNm/m standard webs could be used (for example webs of ϕ8 - 100 mm or ϕ10 - 150). In this situation is chosen for ϕ8 - 100 mm. For the large negative moment no standard webs could be used special webs should be made or individual bars should be connected to each other. The last option is not beneficial because it is very labor intensive.

Table 32 - Required reinforcement

M_{Ed} [kNm/m]	A_s [mm^2/m]	ϕ [mm]	Heart to heart distance [mm]	
			required	Used
-133	2266	16	83	80
20	340	8	142	100

Figure 58 represtens the top view of a gate element of 20 m wide. The bars in the x-direction and y-direction are presented. A detailed presentation of the bars in the gate is represented in Figure 59. Figure 59 representst a cross section of 1 m wide (A-A' in Figure 58) including the of the gate.

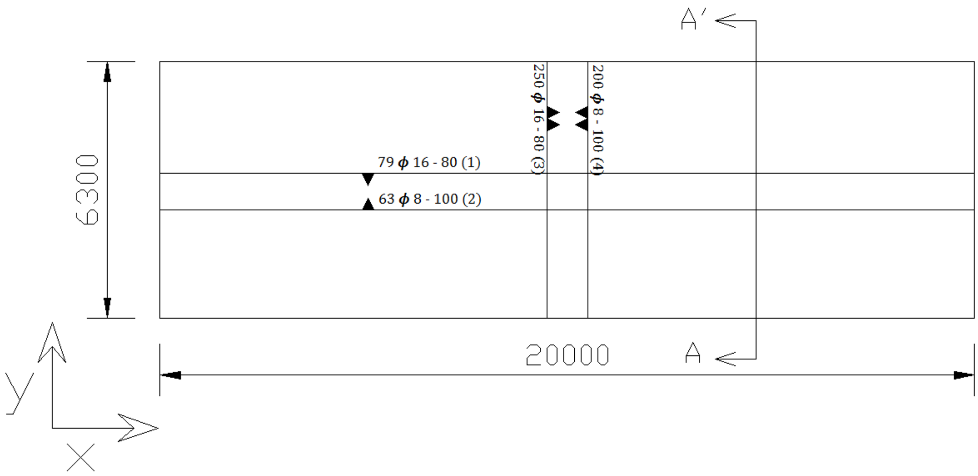


Figure 58 - Reinforcement of the gate

The numbers between the brackets in Figure 58 correspond with the numbers between the brackets in Figure 59.

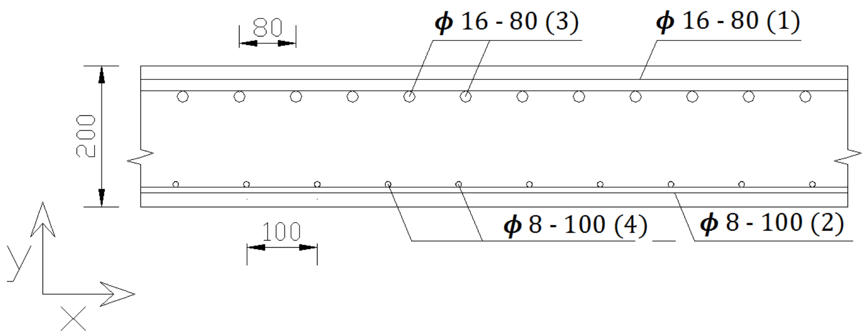
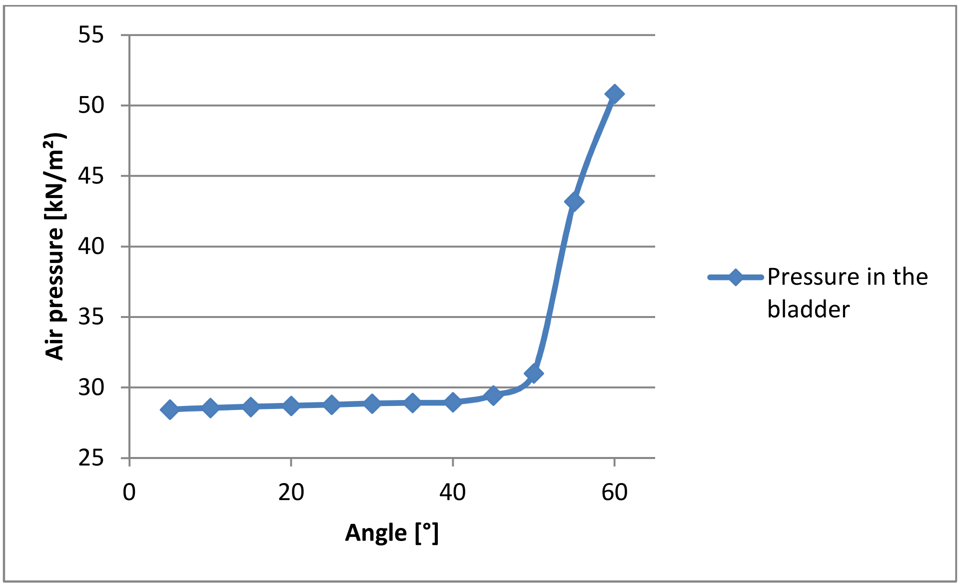


Figure 59 - Reinforcement detail of 1 m of cross section A-A'.

5.8.4 Bladder

As was presented in section 5.7.1.1 resulted the determination of the moment and shear force lines in two variables. The variables were the supporting length of the bladder (contact length between bladder and gate) and the pressure in the bladder. The development of the pressure in the bladder with respect to the gate angle is presented in Graph 4. The transition between the partly opened and closed situation is clearly visible. For the closed condition the pressure in the bladder increases relatively fast. The reason is that the bladder becomes in fully blown up situation, the bladder takes it final shape. When filling the bladder with more the bladder will not reshape, the tension and strain of the sheet material will increases. The largest pressure is obtained for a gate angel of 60°, the pressure in the bladder is for this angel p = 51 kN/m^2. The corresponding contact length between the bladder and the gate is 3.25 m as represented in the right picture of Figure 60.



Graph 4 - Pressure in the supporting bladders

5.8.4.1 Tension in the bladder sheet

As was presented is detailed dimensioning of the bladder out of scope of this report. Further investigation has to be done to the behavior of the supporting bladder and the tension capacity of the sheet material. In this report only a simple check of load the sheet material is done. The load in the membrane is represented in Figure 60. The load in the membrane is represented by the product of the air pressure inside and the radius of the bladder. The magnitude of the load is independent of angle α. This means that the tension loads in the bladder are equal in every point of the bladder. This result is also valid for a closed ring as presented in the middle picture of Figure 60. The right picture represents the supporting bladder. Only the free part of the bladder is considered (not in contact with the weir bottom or the gate). The bladder parts that are in contact with the gate and weir bottom are subjected to friction. Due to the friction the tension loads in the bladder decrease.

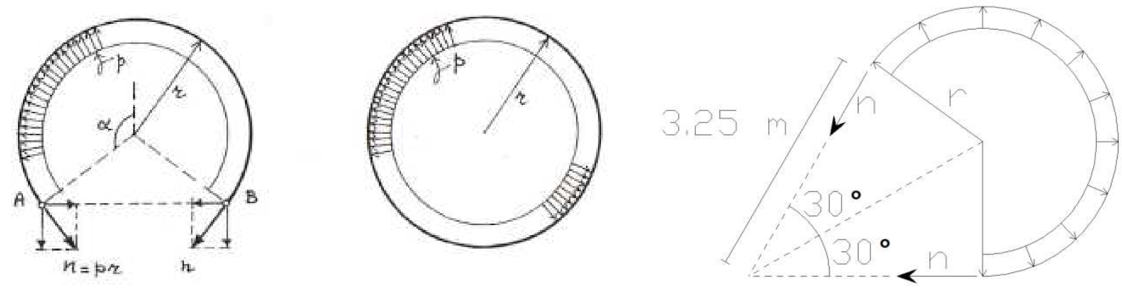


Figure 60 - Tension in the membrane (source: C. Hartsuiker, Toegepaste Mechanica)

The radius of the bladder presented in Figure 50 has a value of r = 1.88 m. The resulting tension load in the sheet material becomes n = 95 kN/m for a maximum air pressure in the bladder (p = 51 kN/m^2).

5.8.4.2 Sheet material

Different sheet materials are available to construct the bladder. Possible materials could be Polypropeen, Polyamide 6 (use name Nylon), Polyester, Aramid (meta-and para-aramid fibers like Kevlar®) or Dyneema® (a high modulus polyethylene (HMPE) fiber). The tensile strength of the mentioned materials are presented in Table 33. The tensile capacities of the membranes are tested according the scale model tests (Bouwdienst Rijkswaterstaat, 2007).

Table 33 - Tensile capacity of membranes

Material	Polypropeen	Polyamide 6	Polyester	Aramid	Dyneema®
Tensile strength [MPA]	600	900	1100	2900	3000
Elongation at break [%]	20	20	13	3.6	3.6

For the sheet material of the inflatable dam of the Ramspol barrier is chosen for Polyamide 6. Polyamide 6 has several advantages like:

- The material has a relative high strength in relation to the elongation capacity compared to Polypropeen
- The material is less sensitive to wear compared to Aramid.
- The lower stiffness of the material compared to Aramid and Dyneema®, the smaller stiffness result is smaller stress concentrations in the membrane.

More investigation should be done to obtain what material in the situation of weir Linne should be the best option. As example for this simple design of the bladder the same material as for the Ramspol barrier is used is Polyamide 6 used because sufficient information about this material is present. The average initial strength of Polyamide 6 in the direction of the circumferential is approximately 1900 kN/m. After fatigue loading, ageing and relaxation from pre-stress the strength of the sheet is reduced to approximately 970 kN/m. The material properties in longitudinal direction differ with the circumferential direction . The initial strength in longitudinal direction is about 450 kN/m.

For the sheet of the Ramspol barrier Equation 62 was used. The factor γ_{mat} represents the material factor, the factor includes all material related aspects. The material factor is derived from a semi probabilistic analysis. The facor γ_{dym} represents the dynamical coefficient . The factor R_t is the capacity of the sheet material, in this case 970 kN/m. The dynamic coefficient are determined with scale models and calculation models.

$$\frac{R_t\,\gamma_T}{\gamma_{mat}} > \gamma_{dyn}\,n$$

Equation 62 - Design check for the sheet material

For the Ramspol barrier γ_{mat} = 1.2, γ_{dym} = 1.3 and γ_T = 1.0 is used. However the situation of bladder of the Ramsmpol barrier is totally different as the bladder for weir Linne. Investigations to the factors should be necessary to obtain a clear check. On the other hand by using the factors for the Ramspol barrier, it is obtained that the right-hand-side of Equation 62 becomes 808 kN/m and the left-hand-side 126 kN/m. The difference between the left- and right-hand-side are relatively large. The factors should be substantially large for failing according to the design check however at according to the check it is not clear if the capacity of the sheet material is sufficient. However it could be possible that other factors should be included or excluded in due to the presence of the flap, more investigation is necessary.

5.8.4.3 Detailed determination

A more detailed determination is the consideration of a infinitesimal small part of the bladder sheet. Every infinitesimal small part has a curvature which creates a balance between the internal pressure, external load, weight of the sheet and the axial membrane force. The balance is presented in Figure 61.

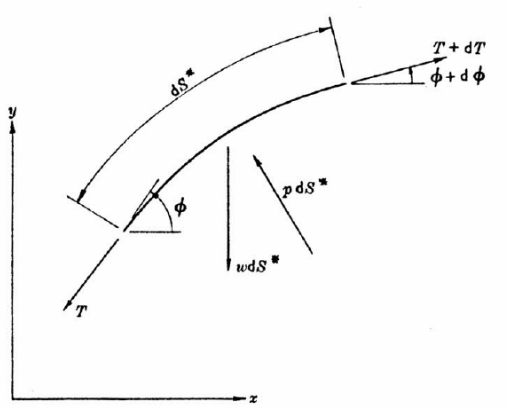


Figure 61 - Balance of a infinitesimal small sheet element (source: Parbery, 1976)

The statically balance of a element with unloaded length dS and loaded length dS* is described for dS* → 0 is presented in Equation 63 and Equation 64. The equations could be solved as a boundary value problem (Parbery, 1976).

$$dT = wdS^{*}\sin\,(\phi_{sheet})$$

Equation 63 - Balance in tangential direction

$$-Td\phi_{sheet} = p_a dS^{*} - wcos(\phi_{sheet})dS^{*}$$

Equation 64 - Balance in radial direction

5.9 Optimization

The gate design in section 5.7 and section 5.8 is based on a straight slab with a length of 6.3 m and a thickness of 200 mm. The gate design could be improved by optimization. Optimization of the gate can be done in several ways. This chapter is focusing on possibilities for optimization of:

- The weight of the gate (section 5.9.1)
- Shape of the gate to increase the discharge coefficient (section 5.9.2)

5.9.1 Weight of the gate

Reduction of the weight of the gate could be a possible option in optimization of the gate. Weight reduction results in a smaller pressure in the bladders and a reduction in costs because less material is used. The weight of the gate could be reduced in two ways, namely:

- Deceasing the thickness of the gate
- Create hollow cores in the gate

5.9.1.1 Thickness reduction

At reducing the thickness of the gate attention should a be paid it to the location of the reducing. According to Figure 53 the maximum moments are located in the lower halve of the gate. It is advisable (when reducing of the thickness is possible) to reduce the thickness of the upper halve of the gate. It should be investigated if the thickness of the gate could be reduced and to which thickness the gate could be reduced. The internal lever arm should be sufficient otherwise more reinforcement is required to withstand the bending moments in the gate. If it is required to reduce the thickness of the entire gate stiffening ribs could be applied to the gate. Hoverer stiffening ribs could become problematic with respect to the supporting bladders.

5.9.1.2 Hollow cores

The weight of the gate could be reduced by creating hollow cores in the gate like is done in hollow-core slabs (Dutch: kanaalplaatvloeren). The hollow cores reduce the weight of the gate but also the shear capacity of the gate at the location of the hollow cores. It is advisable to create hollow cores (if hollow cores are possible) at the location where the shear forces are small. According to Figure 55 are the shear forces in the gate the smallest at the upper end of the gate and around two meter from the bottom hinge. Further investigation is required to determine if it is possible to create hollow cores in the construction.

5.9.2 Shape of the gate

The discharge coefficient has influence on the discharge capacity over the gate as can be concluded from Equation 7. Kindsvater and Carter (1959) presented a form of the discharge equation for either a fully or partially contracted sharp-crested vertical weir. However a flap gate is no vertical gate but is under an angel. The angle and shape of the gate have influence on the discharge coefficient of the gate so correspondingly on the discharge capacity. Three correction factor can be applied to the equation discharge equation, namely (Wahlin & Replogle, 1994):

- *Correction factor for drowned flow reduction*
  Villemonte (1947) found that the equation will also be apply when the flap gate is operating under submerged conditions by including a correction factor for drowned flow reduction
- *Correction factor for angle of the gate*
  It was found that the discharge equation for sharp-crested weir will also apply for flap gate if an appropriate value of the discharge capacity is accurately determined with respect to the gate angle.
- *Correction factor for rounding of the crest*
  Schröder and Turner (1929) give an empirical expression for the increase of discharge due to rounding of the crest. It was assumed that the rounding would have less of an effect on the flow rate as the gate angle was lowered

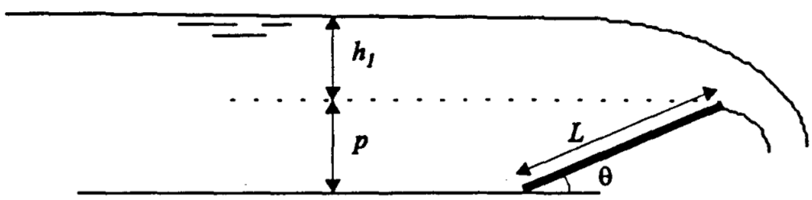


Figure 62 - Determination of the angel of the flap gate in closed condition (source: Wahlin and Replogle)

The formula for the discharge capacity for a flap gate under a certain angle is represented in Equation 7. The discharge coefficient m for a flap gated under angle is represented in Equation 65.

$$m = C_\alpha C_r C_{df} C_e$$

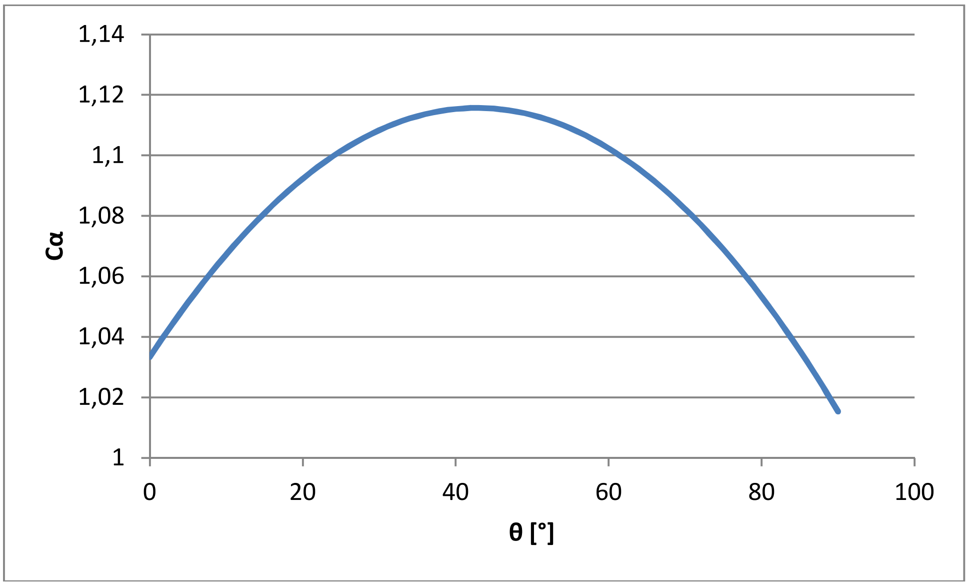
Equation 65 - Discharge coefficient including correction factors

5.9.2.1 Correction factor for angle of the gate

The angle between gate and bottom has influence on the discharge coefficient according to the coefficient C_α. The formula for the coefficient can be simplified to:

$$C_\alpha = 1.0333 + 0.003848\theta - 0.000045\theta^2$$

The formula is plotted in Graph 5 for angles between 0 and 90 degree. It can be seen that the graph is parabolic shaped. By differentiating the formula for C_α and equalizing to 0 is the most preferable angle be calculated. The result is an angle of θ = 42.75°, the corresponding angle coefficient is C_α = 1.116.



Graph 5 - Correction factor for angle of the gate

The length of the gate is determined by the upstream water level and the angle between gate and bottom, an angle of 42.75° results in the highest reduction factor. A high reduction factor is most preferable for small values of h_1 (represented in Figure 62) as can be concluded from Figure 35. The graph (until the transition point) is relative steep for a relative large crest height and relative flat for relative small crest heights.

However an straight gate under a angle of 42.75° results in a gate length of 7.14 m. By curving the gate could the length of the flap be reduced. In closed condition, a straight flap gate usually makes an angle of 60° to 70° with the weir bottom (Boiten, 1995). In lowered position, flap forms a continuous surface with the weir bottom. To maintain the discharge coefficient the crest has to make an angle at of 42.75° with the water surface. The right picture of Figure 63 represents a straight flap with an angle of 42.75° with respect to the bottom. The figure also represents two curved flaps with angles of 60° and 70° with respect to the bottom and an crest angle of 42.75° with respect to the water surface.

Table 34 - Data for flap gates of different curvature

θ [°]	rR [m]	α [°]	Flap length L [m]	Reduction [-]
42.75	-	-	7.14	-
60	14.12	23	5.67	0.21
70	9.04	33	5.21	0.27

The effect of the curvature on the length of the gate is determined in Table 34. A representation of the model is represented on the left side of Figure 63. The flap length for angles θ of 60° and 70° are determined by determining the arc of a circle with radius r. The length of the arc is determined by Equation 66.

$$L = \frac{\alpha * \pi * r}{180}$$

Equation 66 - Length of a curved gate

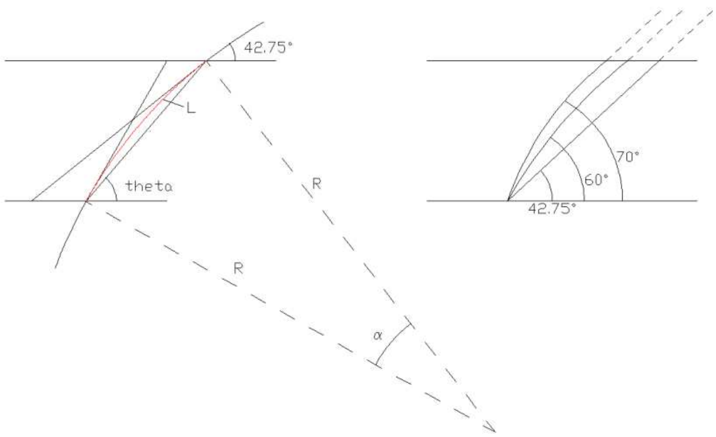


Figure 63 - Curvature of the flap gate

The most ideal angle of the gate with respect to the bottom and the radius of curvature has to be determined in further investigation. For simplification reasons is chosen to design a flap gate with a angle of 60° with the weir bottom.

5.9.2.2 Correction factor for rounding of the crest

The shape of the crest has influence on the discharge of the gate. The radius of the rounding of the crest is governing for the increase of the correction factor as is represented in Equation 67. From Equation 67 can be concluded that the influence of the correction factor is the largest for high angle gates.

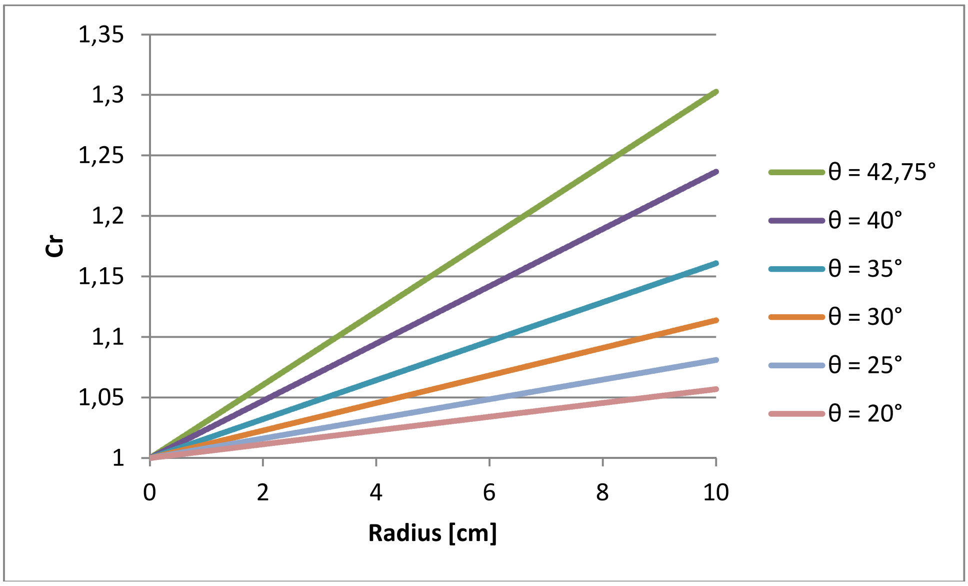
$$C_r = 1 + 0.0368 \frac{r}{\left(h_{up} - h_{cr}\right)^{0.75}} \sin(\theta)$$

Equation 67 - Correction factor for rounding of the crest

where

r = radius of the crest rounding [cm]

The correction factor for the rounding of the crest is represented in Graph 6 for different angles between gate and bottom. As can be seen in the graph is the discharge coefficient the largest for larger angels. As in section 5.9.2.1 is focused on a crest angel of 42.75°. A radius of 5 cm is assumed for the crest of the gate. A corresponding correction factor of C_r = 1.15 is the result for a angle of 42.75°. Further investigation is required for a optimal radius of the gate with respect to large and small angles.



Graph 6 - Correction factor for angle of the gate

5.9.2.3 Correction factor for drowned flow reduction

When the downstream water level affects the flow over the weri, the weir becomes submerged. In this condition, the discharge depends not only on the upstream head but also the downstream water level as was presented in section 4.6.2.2. VIllemonte (1947) developed a useful equation to estimate the discharge over a submerged rectangular sharp-crested weir is presented in Equation 68.

$$C_{df} = A\left[1 + \left(\frac{h_{down}}{h_{up}}\right)^{3/2}\right]^{n}$$

Equation 68 - Correction factor for a gate in submerged condition

$$A = \begin{cases} -0.0013\theta + 1.0663, & \theta < 60 \\ 1.0, & \theta \geq 60 \end{cases}$$

$$n = 0.1525 + 0.006077\theta - 0.000045\theta^2$$

Equation 69 - empirical constants

As was presented in section 4.6.2.2 does the flow become imperfect for a crest height of 17.15 m, this corresponds with a gate angle of 11°. The angle of 11° corresponds with values for the empirical constants of A = 1.052 and n = 0.214. The result is a correction factor of C_{df} = 1.068.

5.9.2.4 Discharge coefficient

The discharge coefficient is a function of the ration between the width of the discharge opening and the width of the approach channel (b/B) and the ration between the upstream water level and the crest height (h_1/h_{cr}). The formula for the discharge coefficient is:

$$C_e = C_1 * \left(\frac{h_1}{h_{cr}}\right) + C_2 = C_1 * \left(\frac{h_{up} - h_{cr}}{h_{cr}}\right) + C_2$$

The values of C_1 and C_2 are coefficient dependent on the ration b/B. The discharge coefficient is presented above is valid for a gate perpendicular to the bottom of the fixed concrete weir structure, the angle θ in is equal to 90°. For this angle is $B_{Poirée}/B_{Meuse}$ = 1 which corresponds with a sharp crested weir. To make the discharge coefficient valid for a gate under angle is Equation 65 used.

By adapting the gate as presented in the sections 5.9.2.1 to 5.9.2.3 the discharge capacity of the gate could be increased. According to Equation 65 becomes the discharge coefficient for a small discharge (perfect weir) m = $1.28C_e$, as presented in Equation 70. The discharge coefficient can be increased by 28% by reshaping the gate as presented in section 5.9.2.1 and section 5.9.2.2.

$$m = C_\alpha C_r C_e = 1.28\left(C_1 * \left(\frac{h_{up} - h_{cr}}{h_{cr}}\right) + C_2\right)$$

Equation 70 - Discharge coefficient for a perfect weir

5.9.3 Acuracy of the flapgate

In chapter 3 was concluded that the accuracy of the current weir is insufficient. The advantage of the inflatable flap gate is that the crest of the weir gate could be set at every required height. Due to this property the accuracy of the weir is increased. The discharge accuracy of the Poirée and Stoney gates were presented in Figure 18. In Figure 64 the discharge of the flap gate is presented. The graph is constructed according to Equation 1 with use of the discharge coefficient determined in section 5.9.2.4. The figure presents a higher discharge accuracy than in the present situation with Stoney and Poirée gates.

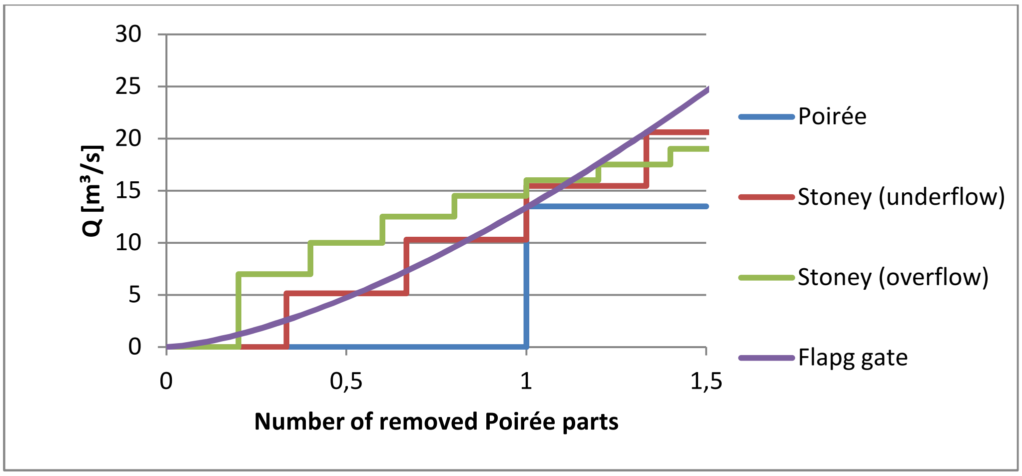


Figure 64 - Acuracy of the flap gate

It has to be noticed that the representation of Figure 64 is valid if the gate does not moves or vibrates. It could be possible that the gate is bouncing a bit on the bladder when water is flowing over the crest. To obtain insight in the effect of the bouncing gate on the bladder to the discharge accuracy should be investigated.

5.10 Placing of the flap gate

5.10.1 Way of construction

For placing the new flap gate it is important the placement takes place between two high water periods. It is advisable to construct the new gates in the summer months, in this period are the Meuse discharges the lowest. As presented in section 2.2.1 is the summer period between June and October, so five months are available to place the gates. However it is decided to construct the gate in sections instead as one construction. In this way it becomes possible to place the gate in more than one summer period. Dividing the gate in sections has more advantages like maintainability or discharge regulation. The number of sections is determined in section 5.5.3.

The new flap gate can be place in two ways, namely:

- *In situ*

In situ refers to construction which is carried out at the building site. In situ techniques are often more labour-intensive, and take longer, but the materials are cheaper, and the work is versatile and adaptable

- *Prefabricated*
  Prefabrication is the practice of assembling components of a structure in a factory or other manufacturing site, and transporting complete assemblies or sub-assemblies to the construction site where the structure is to be located.

Prefabricated techniques are usually much quicker, therefore saving money on labor costs, but factory-made parts can be expensive. They are also inflexible, and must often be designed on a grid, with all details fully calculated in advance. Finished units may require special handling due to excessive dimensions. However are prefabricated (concrete) structures of higher quality than in situ constructed structures. Because the construction time is limited, it is decided to construct the a prefabricated gate. The principle of the prefabricated structure is presented ins section 5.10.3.

5.10.2 Important design aspects

In designing a prefabricated gate construction should several aspects taken into account. Of course are more aspects of importance however not all aspects could be taken into account in this reports The three mentioned aspects taken into account are:

- *Connections between gate structure and bottom slab*
  A prefabricated flap gate should be placed on the Poirée part of the weir. The new flap gate will be placed on the location where the Poirée gate was located. The yokes of the Poirée part were connected to the bottom by hinged connections as presented in section 4.1.1.2. Figure 28 represent a detail of the connection between bottom and yoke. In the step of the bottom, presented in Figure 28, is a notch located wherein the upstream part of the Poirée yoke is connected. The yoke is able to rotate in the notch around the horizontal axis. The distance between two yokes is 4 m so the distance between two notches is 4m. It is decided to connect the new construction with the notches.

- *Storage of the bladder when it is empty*
  In closed condition is the gate in raised condition, supported by the inflatable bladder. However in opened condition is the flap horizontally on the weir bottom. Damage of the bladder should be prevented. When the gate is totally opened the bladder should be stable stored at the weir bottom. The sheet of the bladder should not able to bulge or become detached from the construction in another way because this could result in blockade of the flow over the weir. It is not advisable to store the gate in opened condition directly on empty bladder because it is possible that the sheet becomes in folded position under the gate. When air is pumped in the bladder high tensions could occur in the folded sheet because the bladder cannot reach its natural shape. To avoid folded bladder sheets under the gate it is advisable to construct a recessed floor to store the bladder sheet when the weir gate is opened. For inflatable flap gates does the the gate (in opened condition) function as cover to avoid detaching and bulging of the bladder sheet (Rijkswaterstaat & WL| Delft Hydraulics, 2005).

- *Tubes for the transport of air to the bladders*
  The tubes to transport air from the pumps to the bladders is commonly placed in the flow of the inflatable construction.

5.10.3 Layout

As was represented in section 5.10.1 will a prefabricated gate construction be placed on the Poirée part. As was presented in section 5.5.3 was assumed that the gate will be divided in 3 gate elements. Because the construction time is limited a complete construction is presented. This construction can be totally assembled on the construction side and hereafter places on the Poirée part The gate section that is used to place at the Poirée part of weir of Linne consists roughly out of three parts. The parts are:

- The construction plate (or bottom plate)
- The gate
- The inflatable bladder

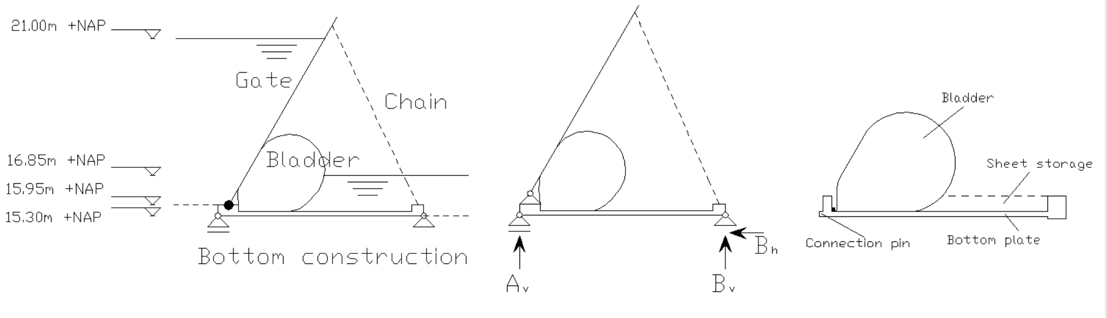


Figure 65 - Flap gate construction

The bottom plate is the construction to which the gate and bladder are connected. The bottom plate contains a deepening which functions as storage of the bladder sheet when the weir gates are open. In opened condition is the gate is resting on the heightened edges of the bottom plate construction, In this way the sheet is safely stored. On the left side of the bottom plate is a construction pin connected. The connection pin is placed in the notches as was presented in section 5.10.2. A detail of the connection between the notch (in the weir floor) and the connection pin of the gate element is presented in Figure 66. The pipes to transport air to the bladders will be placed in the bottom plate. The number of air tubes depends on the number of bladders a gate section contains. As will be presented in section 5.10.4 is the number of supporting bladders

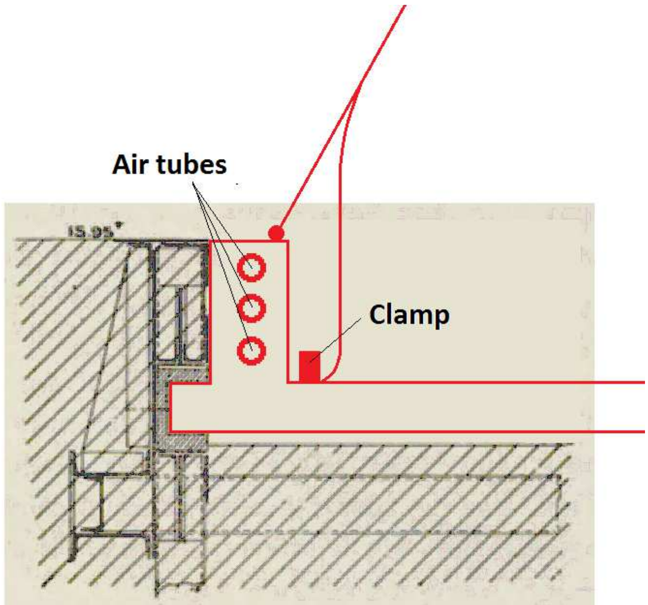


Figure 66 - Detail of the connection between gate construction and notch

5.10.4 Number of bladders

As was presented in section 5.5.2.1 is chosen to use a single bladder instead of two bladders on top of each other. A gate section should be functioning when the weir is closed. The bladder supports the gate section over its full width. If one bladder is used to support the entire gate section will the gate section fail if the bladder fails. To avoid failure of the gate section due to bladder failure, it is advisable to support the gate (in the direction of the width of the gate) by more than one bladder. If one of the bladders fail, the other bladders are able to support the gate. When one bladder fails the gate section will not fail because the other supporting bladders keep the flap in raised position. The number of bladders (next to each other) depends on:

- *Tension capacity of the sheet material of the bladder*
- *Torsion capacity of the concrete gate*

5.10.4.1 Capacity of the sheet material

The tension capacity of the sheet material determines the maximum pressure in the bladder due to external loads. If one of the bladders fail will the pressure in the remaining bladders be increased. It should be avoided that the other bladders fail due to a too high air pressure in the remaining bladders. The pressure will in the situation become higher than presented in Graph 4.However as presented in the introduction is the capacity and dimensioning of the bladder out scope of this report.

5.10.4.2 Torsion capacity

When one of the bladders fail is the gate section not supported over its full width. This results in torsion in the gate. Torsion could result in failure of the concrete gate. In determining the number of supporting bladders this should taken into account. The type of torsion that is exposed in this case is called compatibility torsion. In compatibility torsion, the torsion is induced by the need for the gate to undergo an angel of twist to maintain deformation compatibility, and the resulting twistling moment depends on the torsion stiffness of the gate.
The contribution of the core of the plate with respect the torsion stiffness is neglect able. The torsion stiffness of the gate could be effectively increased by creating a thick stiffening edge around the gate. The stiffening could be created by thickening of the plate edges. In this way the stiffing edge acts (partly) as a kind of box section as is presented in Figure 67.

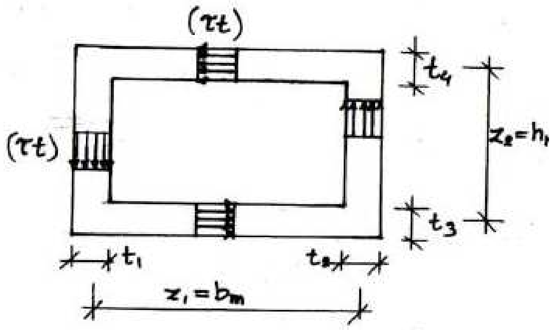


Figure 67 - Constant shear flow in a closed construction

Further investigations are required to determine the optimized number of bladders.

5.10.5 Support reactions of the bottom plate

5.10.5.1 Vertical support reactions

Because the new flap gate will be placed on the old Poirée part will the notches act as connection points. However the loads at the locations of the notched should not be larger than in the current situation because otherwise the bottom slab will probably fail. It is assumed that the support load at the location of the hinge at the new situation (with flap gate) should not be larger than for the old situation (with Poirée gates). A representation of the loads on for both situations is given in Figure 68. The vertical support reaction of support A should be the same for both situation.

The angle of the flap gate differ from the Poirée gate, the angle of the Poirée gate is 7:1 and angle of the flap gate 60° as was mentioned in section 5.5.1. The result is that the water mass G on the gate will change, the definition of load G is given in Table 35.

Table 35 - Loads on the gates

Closed condition	
Loads	**Type**
$Q_1 \coloneqq \frac{1}{2}\rho g h_{up}^2$	Hydraulic load, dependant on the upstream water level
$Q_2 \coloneqq \frac{1}{2}\rho g h_{down}^2$	Hydraulic load, dependant on the downstream water level
$G = \frac{1}{2}\rho g h_{up}^2 \cot(\alpha)$ $G_2 = \rho g h_{down}(W - x_L)$	Vertical component of the upstream hydraulic load, dependant on: • the upstream water level • the angle between gate and bottom
$P_h = p_a x_L \sin(\alpha)$ $P_v = p_a x_L \cos(\alpha)$	Horizontal and vertical components of the load due to the pressure in the inflatable bladder, dependant on: • the length of the contact plane between gate and bladder (x) • the air pressure in the bladder (p) • the angle between gate and bottom
$S = \rho_c g t L$	Self weight of the concrete gate for a thickness of 200 mm.

Table 20Due to a larger G load will also the vertical support reaction of support B change because it was assumed that the support reaction at point A is the same in both situations. The change of load G also results in a different distance between the two supports. The minimum distance between the supports is determined in section 5.10.5.3. However before the support distance for the bottom plate of the flap gate can be determined, should the vertical support reaction of support A be known. In section 5.10.5.2 will the support reactions for the old Poirée gates be determined.

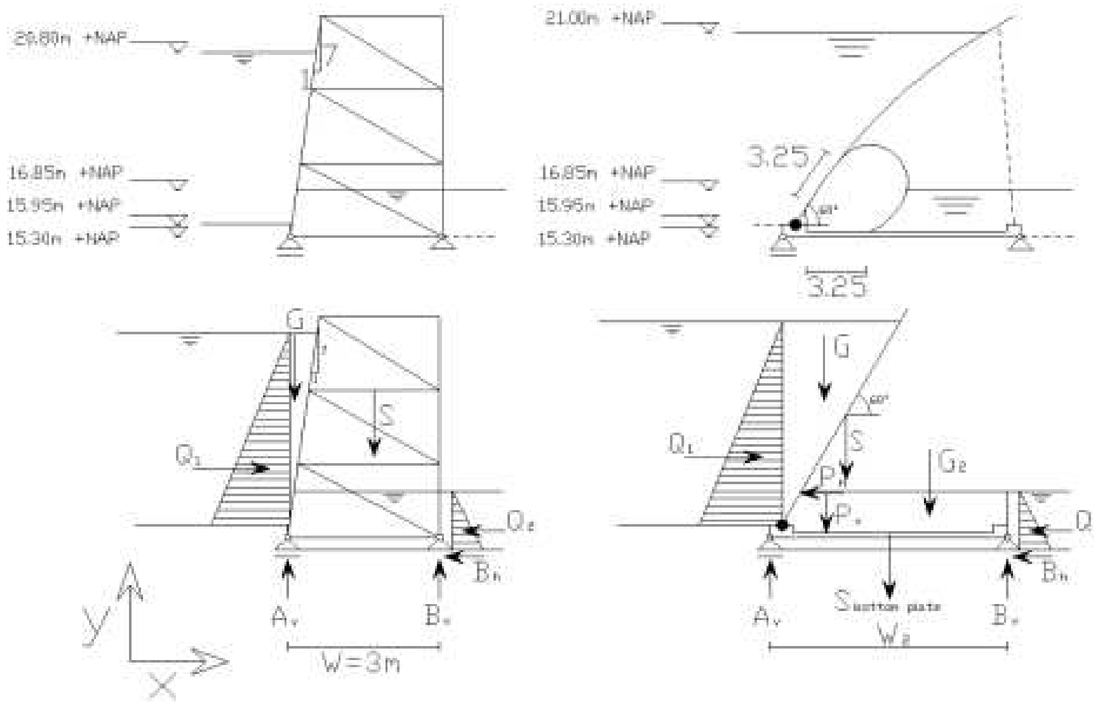


Figure 68 - Load distribution; left: old situation with Poirée gates; right: new situation with flap gate

5.10.5.2 Poirée gate

The Poirée gate is at two points hingely connected with the bottom slab. The support A is located at the notch as is mentioned in section 5.10.2, support B is connected the bottom. Support A is only able to take vertical loads, support B is able to take vertical and horizontal loads. The distance between the two supports of the Poirée yokes is 3 m and the distance between two yokes is 4 m. The support reactions of the Poirée gate are determined according to Equation 24, Equation 25 and with the loads represented in Table 35. The moment equilibrium is taken about support A. The equation for the vertical equilibrium is represented in Equation 71, for the horizontal equilibrium in Equation 72 and the moment equilibrium in Equation 73. The support loads are represented in The loads used in the equilibrium equations are presented in Table 36, also the moment arms between the point of the acting loads and support A are presented in the table. The weight of the yokes and the weight of the partitions of the Poirée weir are not known, in this report a weight of 60 kN for 1 yoke and the associated partitions is assumed. It has to be mentioned that for a exact determination of the support load, the exact weight of the yokes and partitions should be known. In Table 36 are also the support loads represented. These loads are used in the determination of the dimensions of the bottom plate of the flap gate. The dimensions should be chosen that the support load at support A is not exceeding the support load as given in the table.

Table 36, in the middle column are the support reactions per unit width represented. In the right column are the support reactions (as point load) at the notches(support A) and the hinged connected B support represented. As is presented in The loads used in the equilibrium equations are presented in Table 36, also the moment arms between the point of the acting loads and support A are presented in the table. The weight of the yokes and the weight of the partitions of the Poirée weir are not known, in this report a weight of 60 kN for 1 yoke and the associated partitions is assumed. It has to be mentioned that for a exact determination of the support load, the exact weight of the yokes and partitions should be known. In Table 36 are also the support loads represented. These loads are used in the determination of the dimensions of the bottom plate of the flap gate. The dimensions should be chosen that the support load at support A is not exceeding the support load as given in the table.

Table 36 is the vertical support reaction at support A directing in the opposed direction than represented in Figure 68.

$$A_v - G - S + B_v = 0$$

Equation 71 - Vertical equilibrium for the yokes of the Poirée part

$$Q_1 - Q_2 - B_h = 0$$

Equation 72 - Horizontal equilibrium for the yokes of the Poirée part

$$Q_1\left(\frac{1}{3}h_{up}\left(h_{bottom,up} - h_{bottom,down}\right)\right) - Q_2\frac{1}{3}h_{down} + G\frac{1}{3}h_{up}\cot(\alpha) - B_v\,W + S\,\frac{1}{2}\,W = 0$$

Equation 73 - Moment equilibrium for the yokes of the Poirée part around support load B

The loads used in the equilibrium equations are presented in Table 36, also the moment arms between the point of the acting loads and support A are presented in the table. The weight of the yokes and the weight of the partitions of the Poirée weir are not known, in this report a weight of 60 kN for 1 yoke and the associated partitions is assumed. It has to be mentioned that for a exact determination of the support load, the exact weight of the yokes and partitions should be known. In Table 36 are also the support loads represented. These loads are used in the determination of the dimensions of the bottom plate of the flap gate. The dimensions should be chosen that the support load at support A is not exceeding the support load as given in the table.

Table 36 - Loads on the Poirée gate

Symbol	Load	Force	Arm
Input			
Q_1	115 kN/m	460 kN	2.33 m
Q_2	12 kN/m	48 kN	0.52 m
G	17 kN/m	68 kN	0.47 m
S		*60 kN*	*1.5 m*
Result			
A_v		-160 kN	
B_v		465 kN	
B_h		412 kN	

5.10.5.3 Inflatable flap gate

The vertical support reaction of support should not be larger than in the current situation as was represented in section 5.10.2. Due to a different gate angle compared to the Poirée gate will the support reactions changes. The distance between the supports should be chosen correctly to avoid too large support reactions at support A. The support reactions are determined according to Equation 24, Equation 25 and the loads represented in Table 35. The loads due to the bladder (P_v and P_h are not taken into account in the equations because the bladder has no influence on the support reactions of the total flap construction (flap + bottom plate). The loads on the flap gate are presented in Figure 68. The distance 3.25 presented in Figure 68 is the contact length between bladder and gate/bottom. The contact length was presented in Table 23. The moment equilibrium is taken about support A. The equation for the vertical equilibrium is represented in Equation 74, the horizontal equilibrium in Equation 75 and the moment equilibrium in Equation 76.

$$A_v - G - G_s - S - S_{bottom\,plat} + B_v = 0$$

Equation 74 - Horizontal equilibrium for the flap gate

$$Q_1 - Q_2 - B_h = 0$$

Equation 75 - Horizontal equilibrium for the flap gate

$$Q_1\left(\frac{1}{3}h_{up}\left(h_{bottom,up} - h_{bottom,down}\right)\right) + G\frac{1}{3}h_{up}\cot(\alpha) + G_2\left(\frac{1}{2}(W_2 - x) + x\right) + S_{bottom\,plat}\frac{1}{2}W_2 - Q_2\frac{1}{3}h_{down} - B_v * W = 0$$

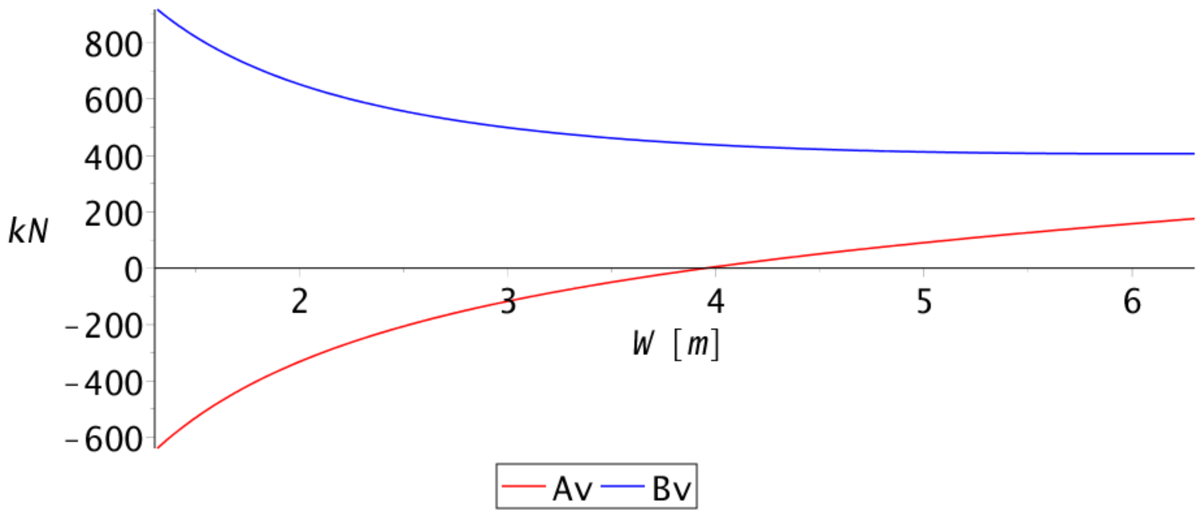
Equation 76 - Moment equilibrium for the flap gate part around support load B

Table 37 - Loads on the flap gate construction

Symbol	Load	Force	Arm
Input			
Q_1	115 kN/m	460 kN	2.33 m
Q_2	12 kN/m	48 kN	0.52 m
G	17 kN/m	68 kN	0.47 m
S	7 kN/m	28 kN	1.58 m
G_2	9 (W_2 - 3.25) kN/m	36 (W_2 - 3.25) kN	0.5(W_2 - 3.25)+ 3.25 m
$S_{bottom\ plate}$	*5 kN/m*	*20 kN*	*0.5 W_2*
A_v		-159 kN	
Result			
B_v		526 kN	
B_h		412 kN	
W_2			2.7 m

In Table 37 the loads on the flap gate are presented. The horizontal loads are compared Table 36 equal, this means that the horizontal support load B_h remains constant. The load G_2 is dependent on the width of the load and the contact length between bladder and bottom plate. The load of the gate S was determined in section 5.7.1.1 and was presented in Table 21. The dead load of the bottom plate is not known, the load depends on the dimensions and the material of the plate. In this report the dead load is assumed at 5 kN/m.

The calculated minimum width of the bottom plate 2.75 m, for smaller widths the support load in A becomes larger than 159 kN. The support reaction B_v represented in Table 37 is the support reaction for a width of 2.7 m. However the width of 2.7 m is too short when it is for example compared to the contact length between bladder and gate/bottom as presented in Figure 68. The bladder would be outside the bottom plate. When the width of the bottom plate is increased also the vertical support loads will change. The development of the support loads with respect to the width of the bottom plate are represented in Graph 7. It can be seen in the graph that for an increasing width of the bottom plate the support reactions decreases. When the width of the bottom plate becomes larger than 3.9 m the direction of support reaction A changes.



Graph 7 - Development of the support loads

A width of 6.3 m is taken for the bottom plat of the flap gate construction. The width of 6.3 m corresponds with the length of the gate. In opened condition the gate covers the storage (represented in Figure 65) for the sheet material. The width is sufficient for the contact lengths between the bladder and bottom as can be seen at the lengths as presented in Table 23.

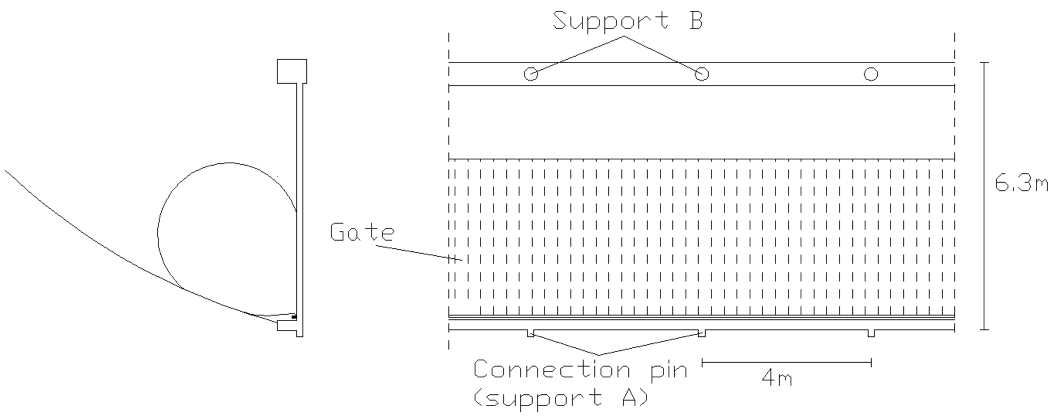


Figure 69 - Top view of a gate element

5.10.6 Horizontal support reaction

As was concluded will the horizontal support reaction not change for the flap gate. As was presented in Figure 68 the horizontal load has to be transported by support B to the concrete weir structure. The capacity of the concrete weir structure should be sufficient to transfer the horizontal loads to the subsoil. Because the magnitude of the horizontal does not change for the flap gate compared to the Poirée gate it could assumed that the capacity of the bottom of the concrete weir structure is sufficient with respect to the horizontal loads on the flap gate. Several connections between bottom and support B are possible like dowels or bolds. The connection should be done after the gate is placed.

5.11 Visualization of the flap gate

In this section the gate elements are visualized to obtain a clear view about the gate element and the elements placed at the current construction. In Figure 70 the front and backside of a closed gate element are represented. The three bladders, curved gate, chains and the bottom construction are represented. Figure 71 represents the gate element in partly opened condition. Figure 72 and Figure 73 represents the gate elements placed on the current weir construction.

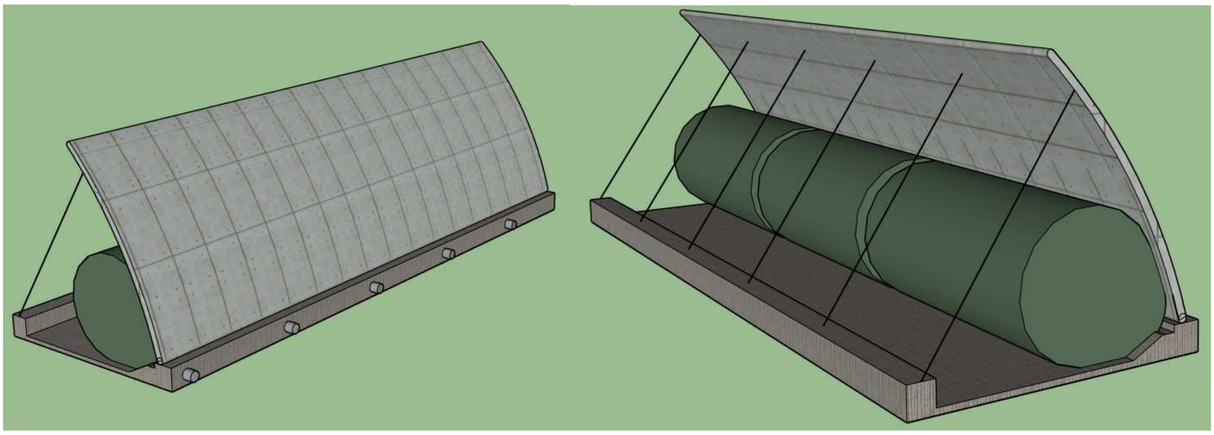


Figure 70 - Gate element in closed condition

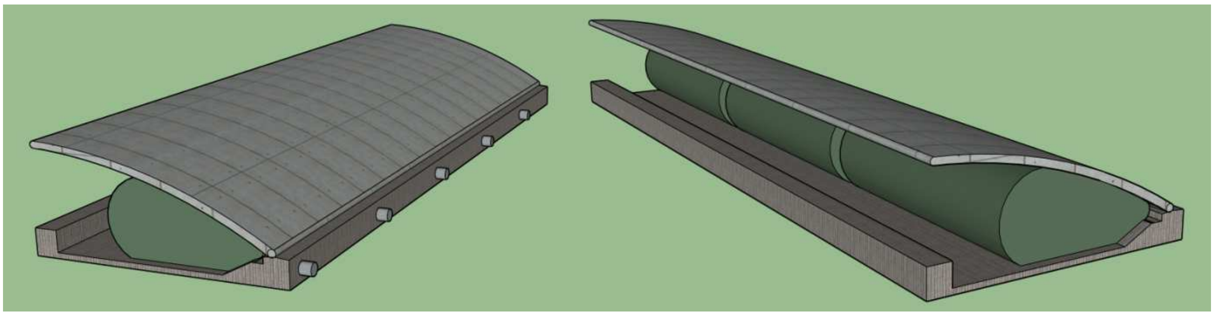


Figure 71 - Gate element in partly opened condition

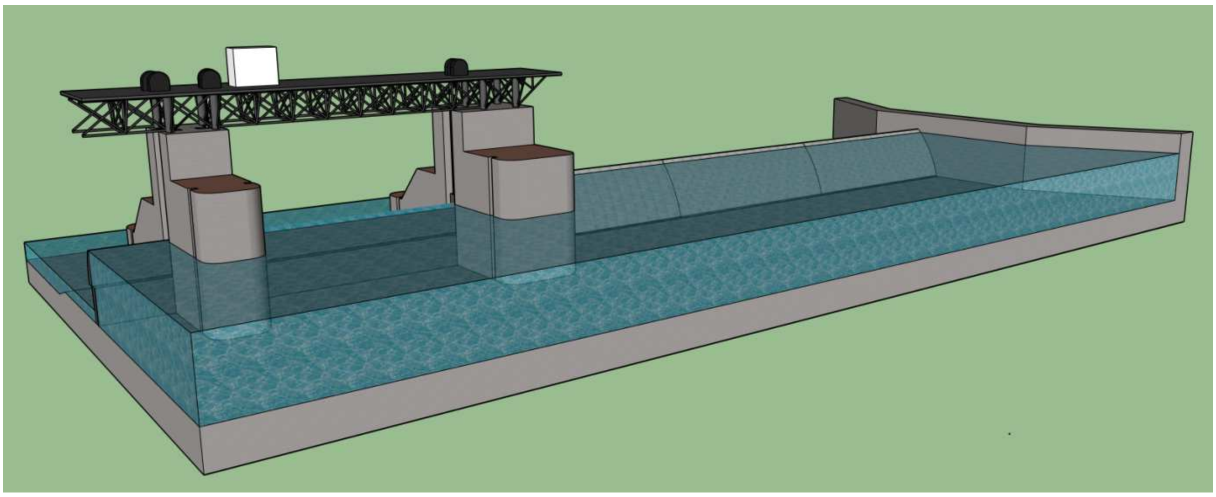


Figure 72 - Gate elements on the current weir (front view)

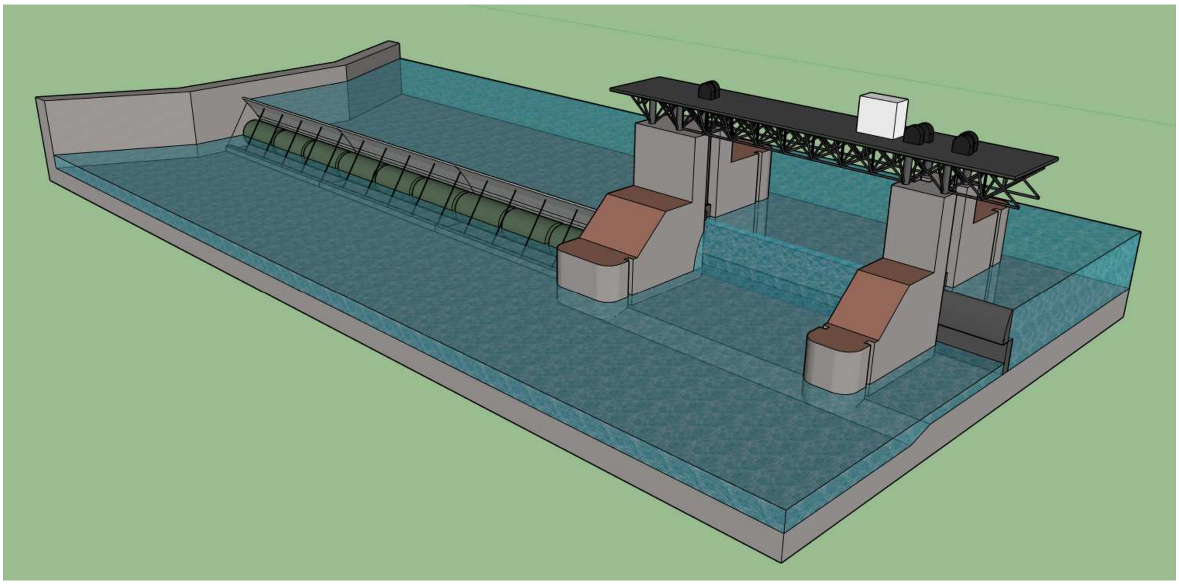


Figure 73 - Gate elements on the current weir (back view)

5.12 Conclusion

The flap gate is designed in concrete class C90/105. High strength concrete as presented in section 5.5.4 is a durable construction material and requires less maintenance than steel, especially in wet environments. The gate makes an angle of 60° with the weir bottom as is generally used for flap gates as presented in section 5.5.1. The gate will be supported by a single air filled bladder and chains to keep the gate fixed to the bladder. The gate will be filled with air instead of water because the water in the bladder could freeze in winter months and sloshing of the water in the bladder is prevented.

Three different load cases are distinguished, namely the closed condition, the partly opened condition and the opened condition. The three load cases are coupled to different gate angles. The load combinations differ in the three situation which results in the moment and shear force lines as presented in Figure 52 and Figure 54. Positive and negative moments can occur in the gate construction. The construction meets the requirements according to Eurocode 2 with respect to the minimum and maximum reinforcement area, the crack width and the moment of failure. The gated does not require shear force reinforcement, the shear resistance of the gate is sufficient. It can be concluded according to the calculations and requirements of Eurcode 2 that a flap gate constructed in high strength concrete is possible for weir Linne. This gate has a length of 6.3 m and a thickness of 200 mm. The thickness could be reduced when optimizing the gate. According to the check of the capacity of the bladder it is not clear if capacity of the bladder sheet is high enough. Investigations are required to obtain a good check formula that includes the effect of the gate.

The gate will be placed as a prefabricated element the element will have a length of 20 m. The gate element contains the gate, bladder, chain and supporting bottom plate. It is advisable to support the gate sections with more than one bladder to keep the gate operable if one of the supporting bladders fail. The plate is placed at the rotation notched of the former Poirée gates. The distance between the support of the bottom plate should have a minimum length of 2.7 m. For a larger width of the bottom plate the load in support A will increase and support B decrease. A with of 6.3 m is chosen because in opened condition the gate covers the sheet storage. The horizontal load of the flap gate should be transferred by the downstream support to the concrete weir structure. The horizontal support load remains constant for the flap gate compared to the Poirée gate.

The gate could be further optimized, optimization could be done in several ways. The gate could be optimized by reducing the weight of the gate due to reduction of the gate thickness or by applying hollow cores into the gate. The discharge capacity of the gate could be optimized by curving the gate with a radius of 14 m, the discharge capacity of the weir could be increased by 28% as presented in section 5.9.2. The accuracy of the weir is improved due to the flap gate, however it should be mentioned that bouncing of the gate on the bladder has influence on the discharge accuracy.

Conclusion & recommendations

Conclusion

In this report was investigated in what manner weir Linne should be improved for the future. The investigation is done at four scale levels.

In the first scale level the Dutch part of the Meuse was considered. In this design level was determined if a different configuration of weirs was possible and if weir Linne should be part of that new configuration. To maintain navigation on the Meuse weirs are necessary. Canal section Linne and consequently weir Linne should be retained because the canal section fulfills an important function for navigation. Combining canal section Linne with canal section Maasbracht results in extensive water level changes. Combining with canal section Roermond is not preferable because changing water levels result in navigable connections and the amount of locks, bridges and harbors in the canal section that should be reconstructed. Other possible combinations of canal sections have no influence on weir Linne.

The second scale level was focusing on canal section Linne . In this section was investigated in which way the weir should behave to high and low discharges with respect to opening of the weir and the accuracy of the discharge. Also the behaviour of the weir with respect to the damping effect of Maasplassen was taken into acount. The damping effect of the Maasplassen of canal section Linne is most preferable and effective during low discharges of the Meuse because navigation is maintained for a longer period. During low periods an accurate discharge regulation is required, the discharge accuracy of the current weir (especially the Stoney gates) are insufficient. The accuracy of discharging Stoney part is 6 m^3/s, which is insufficient for a minimum required discharge of 10 m^3/s at the Border Meuse. The insufficient accuracy results in a slow decrease of the water level in the canal section during discharges lower than 13 m^3/s. The best option to improve the inaccuracy of the weir is to take the inaccuracy into account in the improvements of the weir because situation of the weir should be changed according to the problem statement. The damping effect has less effect on extreme high discharges compared to low discharges. Because the canal section is relatively small the water level will rise relatively quick, more or less lakes have small influence. An opening procedure shorter than 8 hours is reliable after the high discharges are measured at measure point Maastricht St. Pieter. The duration of opening of the weir is determined by the removal of the Poirée part because the removal of the Poirée part is a manual operation.

The third scale level was focusing on the weir construction of weir Linne. In this section was investigated if the weir should be renewed of upgraded and which type of gate should be used in the future situation of the weir. It is not advisable according to the RINK report to replace the total weir of Linne at the moment. It is advised to monitor the condition of the weir until the period 2030 - 2035 and to take after this period a final decision. After this period a decision should be made to replace or maintain the current weir for a longer period. For this report it is decided to maintain the current weir construction. The concrete weir structure a robust construction due to over dimensioning so could eventually fulfill its function for a longer lifetime than was assumed. The width of the current weir of 60 m is sufficient for the discharge capacity and flow velocity behind the weir. The flow velocities will not affect the bottom protection behind the weir.
The Stoney part of the weir is able to fulfill its function in a propper way. However the Poirée part of the weir does not fulfill the requirements with respect to controllability (remote operation) and ARBO legislations. It is advisable to renew the Poirée part of the weir in order to upgrade the weir to a higher state of maintenace. It is concluded according to a MCA analysis that a inflatable flap gate is a good option to replace the old Poirée weirs.

The fourth design level was focusing on the inflatable flap gate. In this section was investigated if it is possible to design a weir gate in high strength concrete.
The designed flap gate is designed in concrete class C90/105. The flap is modeled as a straight plate. According the loads on the gate and the requirements of Eurocode 2 it is possible to construct a concrete gate for weir Linne. The gate is meeting the requirements of minimum and maximum reinforcement area, the shear capacity, the crack width and the moment of failure. The concrete gate itself is checked, the capacity of the sheet of the supporting is checked however the present checks for bladders are related to inflatable constructions like the Ramspol barrier. Which result in a inaccurate check of the bladder material. Therefore it is not clear if the capacity of the sheet material is sufficient. It can be concluded that the construction of the weir gate in high strength concrete is possible however it not clear of the sheet material of the bladder is sufficient to support the concrete gate. The gate could be optimized by reducing the weight of the gate due to reduction of the gate thickness or the by applying hollow cores into the gate. The discharge capacity and weight of the gate could be optimized. The discharge capacity could be optimized by curving the gate, the discharge capacity of the weir could be increased by 28%. The weight of the gate could be optimized by including hollow cores in the gate of decreasing the gate thickness (if possible).

In this design level it is also be investigated in which way the flap gat should be placed on the current weir structure. A gate will be placed as a prefabricated element. The gate element contains the gate, bladder, the chain and supporting bottom plate. The gate is not constructed as one element but divided in 3 elements of 20 m wide. It is advisable to support the gate sections with more than one bladder to keep the gate operable if one of the supporting bladders fails. The plate is placed at the rotation notched of the former Poirée gates. Because the gate elements are places at the rotational notched the upstream support is able to transfer only vertical loads. The horizontal loads on the gate should be transferred by the downstream support. The gate could be further optimized, optimization could be done in several ways. The accuracy of the weir is improved due to the flap gate, however it should be mentioned that bouncing of the gate on the bladder has influence on the discharge accuracy.

Recommendations

Chapter 2

It is concluded that canal section Linne could be maintained for the current layout of the Meuse corridor. A combination with upstream or downstream canal sections is not the best option. However it holds for the current layout of the Meuse corridor. If a total new layout is proposed for the canalization of the Meuse it should be analyzed if weir Linne should be maintained or removed. It has to be investigated if weirs could be placed at other locations than in the current configuration. Other configurations should be investigated with respect to flood protection, bridges, harbors, locks and other factors.

Chapter 3

The effect of the Maasplassen is high for low discharges and low for high discharges. Relative to each other the variants do represent a clear view about the effect of the Maasplassen on the water level changes. It has to be noticed that the results of the behavior analysis as presented in chapter 3 do not represent the real situation. In the analysis the banks and flood areas are omitted in the analysis of the effects of the Maasplassen to the water level changes in canal section Linne. The velocities of the water level changes will in the real situation be smaller. Investigation has to be done to the real effect of water level changes in the canal section during varying discharges.
The duration before a flood wave reaches Linne is assumed at 8 hours. Investigations on running times of flood waves on the Meuse have been done but are not valid any more. Due to the Grensmaasproject running times for flood waves will change. Investigation should be done to

running times of flood waves on the Meuse after the situation when the Grensmaasproject is finished.

Chapter 4

In this report is it assumed to maintain the current weir construction and replace the Poirée gates by an inflatable flap gate. Proper investigation is necessary to determine if maintaining the current construction is the best option. Due to monitoring until 2030 - 2035 more information could be obtained about the construction.
The dimensions of the discharge opening of the Poirée part are checked to obtain insight if the discharge width of the Poirée part should be increased or could be decreased. Because check is done according to as simple method it is hard to conclude if the discharge width should be increased or could be decreased. More investigation to the discharge width of the weir should be done.

Chapter 5

The checks of reinforcement, cracks and moment of failure are done for bending moments and shear resulting of loads perpendicular to the gate. Torsion due to for example failure of bladders is not taken into account. The concrete gate should be checked to moments, cracks and failure due to torsion moments.
In this report the gate and the bladder were analyzed. The check of the bladder capacity is not accurate because the present checks for bladders are related to inflatable constructions like the Ramspol barrier. More detailed checks should be obtained to check the bladder. This is an important investigation because the strength of the bladder is one of the important factors to functioning of the weir gate.

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