DELFT UNIVERSITY OF TECHNOLOGY

Assessing and redesigning Valkenburg's flood risk management system

A multidisciplinary project

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DISCLAIMER : This is an educational project completed by MSc students of TU Delft within limited time of 8 weeks, based on partial and limited information. This implies that the results in this report are not necessarily fully representative and realistic of the actual situation. To identify feasible and realistic flood risk management strategies for the investigated region, more studies, further analyses and better information are needed.

Abstract

The geographical features in the Southern part of Limburg forces precipitation from upstream located areas to flow through a bottleneck, which is exactly located at the city centre of Valkenburg. This makes increasing the safety level more complicated than in other areas. The safety level of Valkenburg has a lower standard in comparison to the rest of the country, namely 1 in 25 years. The combination of those two characteristics is not desirable. Official documents state that this lower standard is based on detailed (societal) Cost-Benefit Analyses. In reality however, the safety standard is based on simple back of the envelope calculations. The Limburg Waterboard has indeed developed a Cost-Benefit tool which they could use to find out whether the implementation of safety measures are cost effective, however they have not been able to implement it until now. Additional safety measures to increase the safety level are assumed too costly based on the same brief calculations. It is doubtful whether individual risk laws are met, since the Limburg Waterboard assumes no casualties in the Geul area. The 2021 flood however showed that this might be false for future floods which get more severe over time due to climate change.

The citizens and entrepreneurs in Valkenburg were not completely aware of the risks they were exposed to and their sense of safety related to flooding decreased after the flood. Most of the people questioned in a survey demanded a higher safety level than the current standard. They would even be open for an increase in tax to realise this improvement. Raising the quay walls would be a cost-effective solution according to some of the citizens. However, the entrepreneurs who rely on tourist based income, do not prefer this option due to loss in aesthetic value.

Hydraulic, structural, and non-technical solutions which are investigated in this report, have the aim to increase the safety level or make the safety level more acceptable for citizens. The hydraulic, and structural solutions focus on four main aspects. The first aspect is related to the redesign of bridges in the city centre. This is mainly done by applying a flat bridges design, which is further elaborated with a case study for the collapsed Emmalaan bridge, and a liftable bridge design. The second aspect is related to closing the gaps in the quay walls, and increasing the height of the quay walls. The third aspect is related to the implementation of water tunnel concepts with six different design concepts. The fourth aspect is related to implementing parts of Meerssen's 4-step approach. The first three aspects of the hydraulic and structural solutions are focused on increasing the discharge capacity of the Geul, while the latter aspect focuses on retaining, delaying, and storing the precipitation. Non- technical solution are also proposed that focus on making people more aware of the risk they are exposed to. This could eventually lead to more acceptance and thus more pleased citizens.

The first order estimations for investment costs and safety level for the hydraulic, and structural solutions are graphically displayed in order to provide an overview of possible interventions to the municipality of Valkenburg and the Limburg waterboard. Although preliminary, and based on limited available data, these results should encourage both stakeholders, and other relevant parties, to reconsider safety standards and search for measures that could increase the safety level of Valkenburg when desired.

Contents

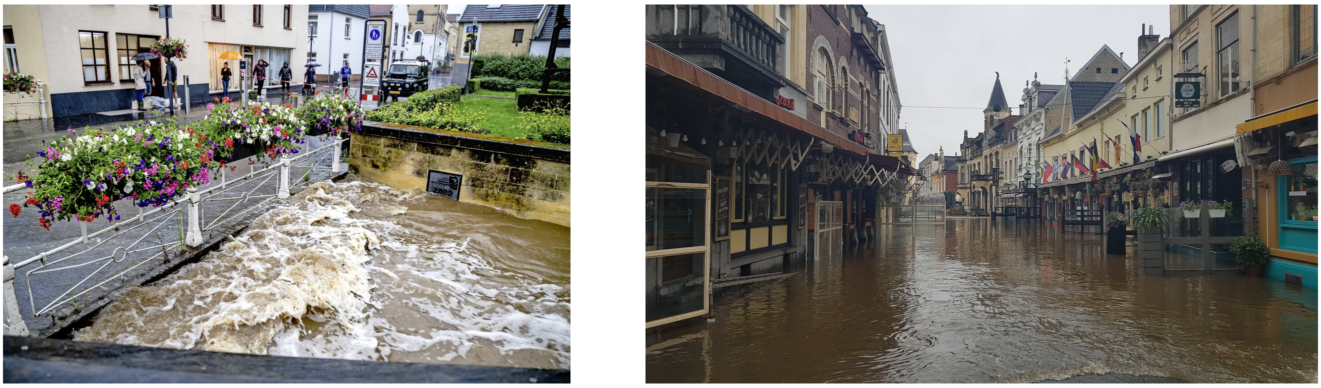
List of Abbreviations

ALARP = As Low As Reasonably Practicable
CBA = Cost-Benefit Analysis
cdf = cumulative density function
FEA = Finite Element Analysis
GEV = Generalised Extreme Value
LC = Load Combination
pdf = probability density function
RWS = Rijkswaterstaat
SLS = Serviceability Limit State
SSM = Schade Slachtoffer Module
ULS = Ultimate Limit State
VRLN = Veiligheidsregio Limburg-Noord
WSS = Waterschadeschatter
WTP = Willingness To Pay

1. Introduction

1.1 Background information

On Tuesday and Wednesday 13 and 14 July 2021, parts of The Netherlands, Belgium and Germany flooded due to extreme precipitation events in the in the Rhine and Meuse catchments. This event caused the discharges in the River Meuse to be up to 3260 m^3/s at St. Pieter (Watermanagementcentrum Nederland, 2021), with extreme discharges occurring in the River Geul as well. These extreme discharges subsequently led to floods in Valkenburg, damaging houses, cars and even destroying a complete bridge. A visualisation of these events can be seen in figure 1.1. Damage estimates range from 100 million euros (Telegraaf, 2021) to 400 million euros (NOS, 2021).



(a) The water reaching the bottom of a bridge in Valkenburg (Telegraaf, 2021)

(b) City center of Valkenburg during the flood (Erfgoedstem.nl, 2021)

Figure 1.1: Pictures taken of the flood in Valkenburg

The topography of Valkenburg can be seen in figure 1.2 a, with the layout of the Geul through Valkenburg shown in figure 1.2 b. As can be seen, the Geul splits into two before entering the city centre.

(a) Overview of area

(b) Valkenburg close-up

Figure 1.2: Map of Valkenburg (Arcgis, 2021)

It cannot be considered a coincidence that Valkenburg flooded. Valkenburg is prone to flooding due to its location in a valley, as can be seen in figure 1.3. In this figure it can clearly be seen that Valkenburg is located in a valley, through which the river Geul flows. This causes all precipitation from upstream to flow through Valkenburg which can result in substantial water levels. Also, the precipitation which falls in Valkenburg itself will flow towards the river Geul as this is the lowest location in the valley. The river is situated in the heart of the city surrounded by built-up area, which makes it a relatively high risk area.

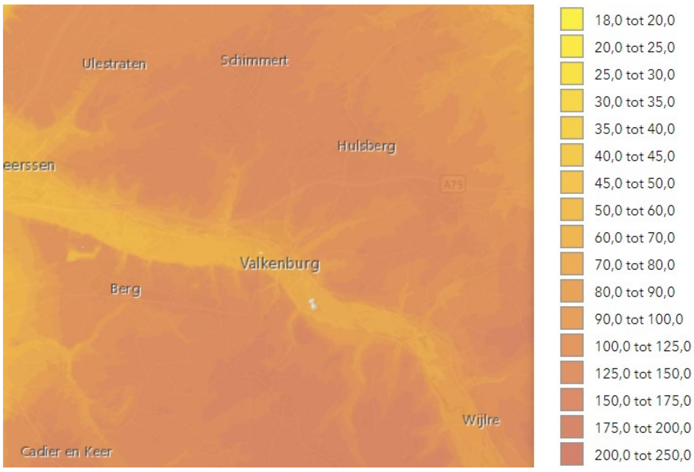


Figure 1.3: Height map of Valkenburg (Algemeen Hoogtebestand, 2021)

1.2 Problem statement

The floods in Valkenburg led to damage estimations up to 400 million euros (NOS, 2021). With this amount of damage, it is no surprise that the municipality thinks it can take up to two years for the area to fully recover (René Willems, 2021). Not all damage was covered by insurance, and the claims that did come through, take a long time to be handled. Due to this, and the floods disrupting the whole of local society, many individuals from Valkenburg are scared to experience floods again in the future. A local entrepreneur said the following on the national NPO radio 1: "No-one will be able to handle this twice. (..) It is impossible to handle this mentally again"(NPO Radio 1, 2021).

The recent floods initiated discussions on whether the current safety standard for Valkenburg is enough. The citizens of Valkenburg seem not in line with the current flood safety standards and it might be considered to include the view of these important stakeholders into new safety regulations. However, Valkenburg is extremely prone to flooding due to its location which makes it very costly to increase its water safety standards.

1.3 Objective

The objective of this paper is to tackle the problems by analysing the choice of safety level in the city of Valkenburg, and redesigning the flood risk management system using different solutions coupled with certain safety levels. This raises the question whether redesigning the river Geul/Valkenburg system is economically feasible to prevent future floods, and if so, what are some feasible designs. Therefore the research question of this project is:

How can the flood risk management system of Valkenburg be redesigned in order to improve the overall safety level?

Within this main frame, the current safety standards are compared to those preferred by different stakeholders like the municipality and the citizens. The safety level will also be compared to other flood defense systems in The Netherlands. The current standards are further analyzed by assessing the damage of the previous floods and applying other, e.g. more innovative measures which may increase the flood safety at relatively low costs. The range of applicability of those measures (costs and possible safety levels) will be indicated.

2. Methodology

2.1 Method of research

With the goal and research question of this report clear, a method is needed to reach these objectives. The report will be roughly divided into two parts: a section concerning the assessment of the current situation and a section with conceptual designs. The assessment of the current situation will be done by means of literature (among other laws, Cost-Benefit Analyses, news items, etc.) and interviews. The Cost-Benefit Analyses of Valkenburg plays an important part and will be updated with the newest information available. This might give insights into the feasibility of new measures. Interviews will be conducted with the water board of Limburg to get a better understanding of the effects of the recent flood as well as the reasoning behind the current safety level. Surveys will be conducted on the residents of Valkenburg to get their opinion on the current and desired future situation.

The conceptual design will take place following these steps:

- Produce conceptual designs
- Elaborate on designs and practicalities
- Estimate costs
- Estimate effect on safety level

The first step will be done by looking at reference projects while taking into account the reasons for the susceptibility of flooding. Then, the range of applicability concerning costs and safety levels will be obtained for each measure whilst taking into account the wishes and restrictions of the waterboard and citizens. This will result into a graph displaying safety levels of different solutions and their investment costs.

Survey

As stated above, a survey will be conducted to get the opinion of the citizens of Valkenburg. This survey plays a big role in this research as the goal of this report is to come to a better solution taking into account not only the wishes of the waterboard, but also those of the citizens. The way this survey will be held is in person, during a field visit to Valkenburg. The contents of this survey will focus on their experiences during the recent floods, the knowledge and wishes with regard to the safety level of Valkenburg and points of view on possible mitigation measures.

2.2 Built-up of report

As stated, the first part of the report focuses on assessing the current situation with the available information while the second part focuses on providing conceptual designs to tackle the problems assessed in the first part. By dividing the main research question into sub questions, a structure can be given to the report. The following sub questions will be answered:

1. Assessing the current flood safety level in Valkenburg.
    (a) Where does it come from?

(b) How is it calculated?

(c) How does it compare to other municipalities?

2. What are different solutions for increasing the safety level of the city of Valkenburg?

3. What are the costs and effects of the different solutions?

3. Analysis

In this section, stakeholders and their needs are identified after which the current flood safety level of Valkenburg will be analyzed. Furthermore, the current state of the water defence systems along the Geul will be analyzed. This information will be used as boundary conditions for the conceptual designs.

3.1 Stakeholder analysis

To find different safety levels for different stakeholders, we first need to find what the attitude of the stakeholders is towards increasing the safety level.

Table 3.1: A summation of the stakeholders

Stakeholders	**Information**	**Role**
National Government	Make laws and regulations for provinces to uphold	Acquaintance
Province	Have obligations to make water safety standards on a regional level	Saviour
Municipality	Keep the city and their inhabitants safe from floods, as well as attract tourists	Saviour
Waterboard	Take and design water safety measures to reach the standard, and to collect waterboard taxes	Saviour
Research companies/ Universities	Find and create innovative solutions to prevent flooding	Friend
Engineering Companies	Implement and design solutions to prevent flooding	Friend
Locals	Limit burden and damages caused by the floods	Friend
Entrepreneurs	Limit burden, damages and suspension of business activities caused by the floods	Friend/ Irritant
Farmers	Limit burden, damages and suspension of business activities caused by the floods or preventive measures. Want fair compensation if their land is used as a buffer.	Irritant
Tourists	Do not want to see flood defences, unless aesthetically pleasing	Trip Wire / Time Bomb
Insurance companies	Want to pay out as little as possible	Friend/ Irritant

In table 3.1, there is an overview of the stakeholders and their roles. For an explanation on the different roles, see figure **??**. The roles are determined by their power, interest and attitude *towards increasing the safety level in Valkenburg*. It is important to take this into account when looking at the table, as the roles of the stakeholders change when looking at different aspects of the floods. Some stakeholders have multiple roles. For example, entrepreneurs have the roles 'friend' and 'irritant'. This is because entrepreneurs want to prevent their business from flooding, but a lot of entrepreneurs in Valkenburg rely on tourism. Therefore, most of them do not want elaborate changes to the old city center in order to keep the tourism sector intact. Essentially the same reasoning goes for the Tourists. They do not want to see the flood defences, unless they contribute to the beauty of the city.

The interests of the insurance companies is to keep payouts as low as possible. The insured people and businesses of Valkenburg are insured for "Vertical water" i.e. rain and sewage water. Some insurance companies now argue that the damage is caused by "horizontal water", or flooding of the Geul, while it is a combination of both (De Volkskrant, 2021). The insurance companies are between 'friend' and 'irritant', because they likely have a neutral attitude towards increasing the safety level, as they will simply adjust their insurance rates and get a similar profit model.

3.2 Assessing the current flood safety level

The current safety level of Valkenburg first needs to be assessed in order to determine the desired safety level for different stakeholders. How was the current safety level determined and is it an appropriate safety level?

Article 2.8 of the general water law, which was introduced back in 2009, states that it is the province's obligation to set standards for the average probability of flooding for areas close to regional rivers (Waterwet, 2009). For Limburg, these standards are set in the *omgevingsverordening* (Provincie Limburg, 2014). The standards are expressed in terms of average probability that the water level rises above surface level (Ministry of Infrastructure and Water Management, n.d.). According to Provincie Limburg, 2014, a safety level of 1:100 is assigned to urban areas, except for the ones located in valleys, like Valkenburg, or other areas where a 1:100 safety level is not possible or not very expensive to achieve. For these places, a safety level of 1:25 is assigned. However, this value can be adjusted when a cost- benefit analysis would indicate it is appropriate to do so.

The recent flood of July 2021 was classified as a 1/100 to a 1/1000 event in Expertisenetwerk waterveiligheid (ENW), 2021. In figure 3.1 the flood area of the recent flood of Valkenburg can be seen. Comparing this to figure 3.2a and 3.2b, it can be seen that the occurred flood area more closely resembles the flood area of that of a 1/1000 event, or possibly even an event with less probability of occurrence.

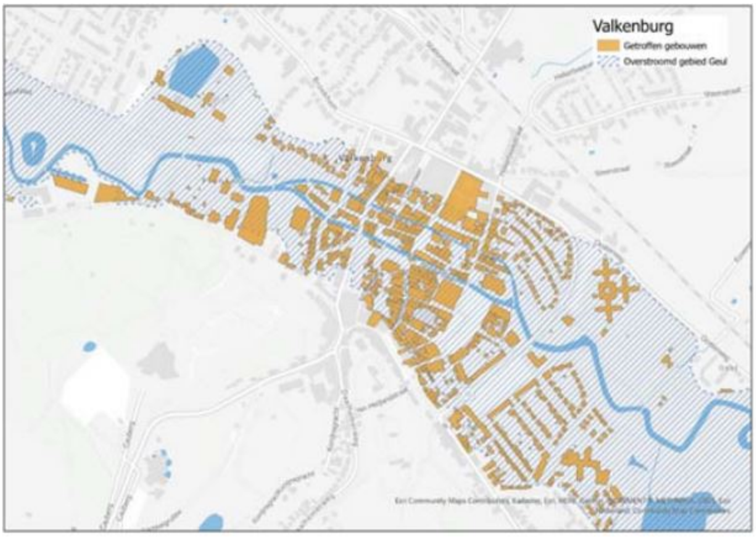
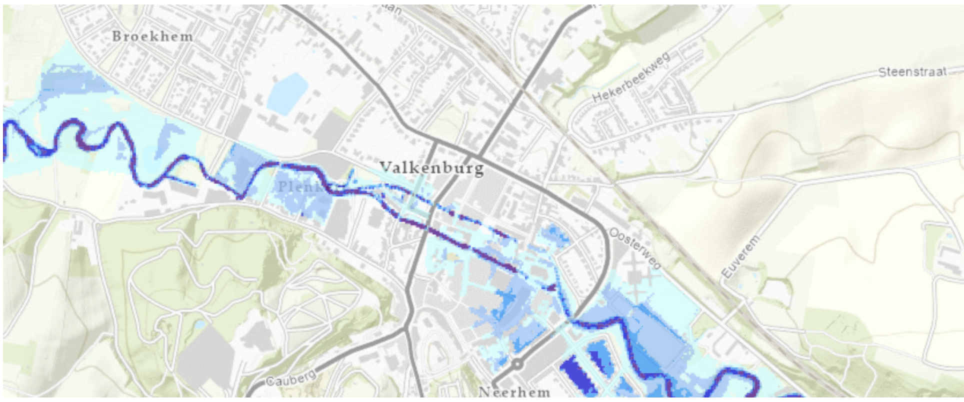
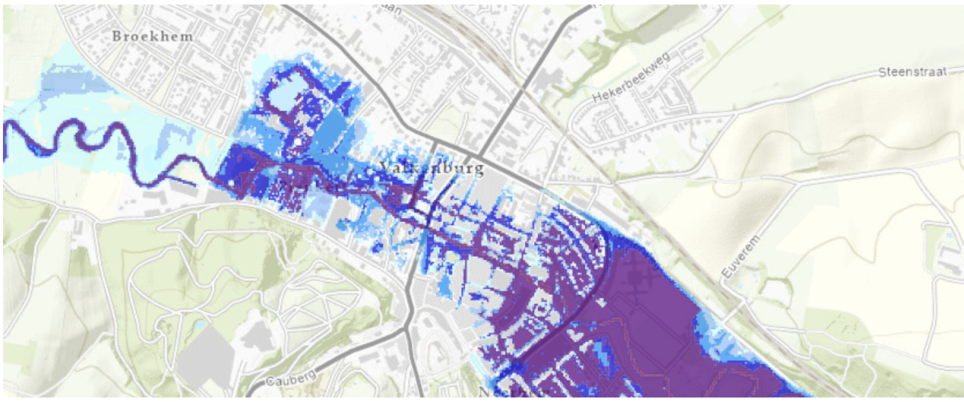


Figure 3.1: Flooded area in Valkenburg (Expertisenetwerk waterveiligheid (ENW), 2021)



(a) Return period of 100 years



(b) Return period of 1000 years

Figure 3.2: Estimated flood area of Valkenburg for two events with a different return period (Risicokaart, 2019)

The Overstromingsrisico's in Nederland Stuurgroep water, 2018, is a document indicating the potential flood risk locations in the Netherlands. Risk locations are classified in the document as locations with potential damages greater than 40 million euros, or whenever deaths will occur. In this document, the Geul is identified as a potential flood risk location, due to its high potential damage (25-50 million euros), and chance of casualties (1-5). The Geul is one of the only five regional water systems that have been identified by Stuurgroep water, 2018, as potentially risky.

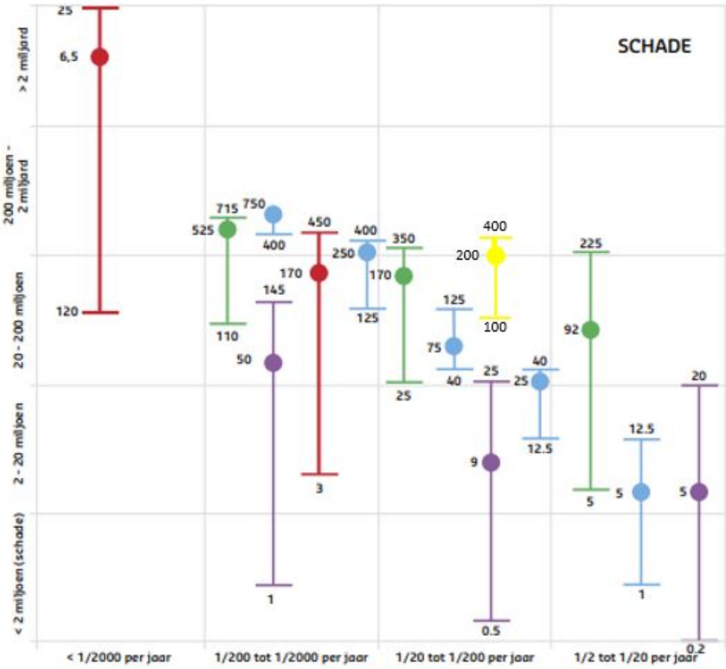


Figure 3.3: Risk diagram of potential damages for different types of floods (Stuurgroep water, 2018). red: primary water defence systems(type B), blue: regional water defence systems (Type C), green: unprotected primary bodies of water (Type A), purple: unprotected regional bodies of water (Type D), yellow: the damage to Valkenburg done by the recent event

In figure 3.3, the safety levels for different types of floods, with their corresponding potential damage can be seen. The Geul is categorized as an unprotected regional body of water, which corresponds to purple in the figure. Knowing that Valkenburg and other villages along the Geul have a safety level of 1:25 (see figure **??**), the expected economic damage would amount to approximately 9 million euros, with an upper limit of 25 million euros. These numbers are already lower than the potential damage according to Stuurgroep water, 2018. However, according to Expertisenetwerk waterveiligheid (ENW), 2021, the actual economic damage is estimated to have been between 250 and 400 million euros along the river Geul. The economic damage thus is significantly higher than what is to be expected for similar floods, and might be an indication that the current safety level needs to be reassessed.

3.2.1 ALARP and Cost-Benefit Analysis

To assess and determine safety standards, the UK makes use of the *As Low As Reasonably Practicable* (ALARP) principle which requires the responsible decision-makers to reduce the risk for society as long as costs are not in gross disproportion to benefits (Jones-Lee and Aven, 2011). *Reasonably practicable* in this principle means that risk could be reduced under most circumstances, but at some point, further risk-reduction is increasingly costly to implement. But what exactly does *gross disproportion* entail? From the theoretical perspective of welfare economy, any measure that leads to greater costs than benefits will lead to a loss in societal welfare. Benefits should therefore exceed costs in order to add welfare. However, this only holds when both costs and benefits are properly measurable. When welfare is added to society, and both costs and benefits are properly measured, we can say that costs are not in gross disproportion to the benefits. Most of the time, these costs can be measured properly because contractors can give accurate numbers on how much a certain safety measure would cost to implement. Some benefits however, may be hard to grasp in terms of a monetary value.

The method that is used to find the ratio between all costs and benefits belonging to a certain safety measure, is called a *Cost-Benefit Analysis* (CBA). The fundamental idea of the CBA is that if the costs of the safety measure are smaller than the induced benefits, and thus costs are not in *gross disproportion* to the benefits, the measure would normally be worth introducing. Vice-vers, if the costs are greater than the induced benefits, and thus the costs are in *gross disproportion* compared to the benefits, it is not worth introducing the safety measure. Jones-Lee states that widespread agreements have been reached about the fact that benefits should be defined in such a way that the preferences of the people affected by the safety measures are strongly reflected. One way to include this preference could for example be the *Willingness To Pay* (WTP) method. The CBA method as described above, is also used by the Province of Limburg.

According to the *omgevingsverordening*, the CBA performed by the province of Limburg showed the policy-makers that the general standard of once in a hundred year is not achievable according to the ALARP principle. This is due to the geographical characteristics of the area as can be seen in figure 1.3. Costs would be in gross disproportion to the benefits when this standard would be striven for and therefore decision-makers have chosen to deviate from the standard. In the new standard for Valkenburg, the probability for flooding is 0.04: on average once in twenty-five years. To answer the question *why* the general standard is not achievable, the actual CBAs have to be assessed. The cost part of a CBA consists of the investment costs and the maintenance over the corresponding years. The benefits that are included in a CBA are however not that straightforward. To properly assess the CBA from the Limburg Waterboard, we need to place it into perspective. This is done by making an analysis of a CBA retrieved by *Rijkswaterstaat* (RWS). After both analyses, we can point out certain differences if present.

Cost-Benefit Analysis Rijkswaterstaat

The CBAs retrieved from RWS, are the ones made for primary levees in The Netherlands. The corresponding level of protection can be found in figure **??**. Looking at the method used to perform the CBA, the costs of flooding have been determined according to the *Schade Slachtoffer Module* (SSM), translated *Damage Victim Module*. These saved costs, and thus benefits, include the following aspects (Deltares, 2011):

- Monetary value of damages related to people
  - People affected
  - People killed
- Monetary value of damages related to other facets
  - Real estate
  - Movable assets
  - Suspension of business activities
  - Indirect damages
    - Loss of revenue for businesses outside the flooded area
    - Loss of travel time

The SSM only indicates / calculates how many people are possibly affected and killed. The monetary value attached to these numbers, are derived from previously performed research by De Bruijn and Van der Doef, 2011. The monetary value for an affected person is set at €12.000. This value includes costs for lost items with emotional value, losses in income, temporary discomfort, personal costs for evacuation, etcetera. The monetary value for a casualty is set at €6,700,000. This value is assumed because it is derived from the *value of a statistical life* in before mentioned literature. The fraction of evacuation is an important parameter in limiting the number of affected people and casualties. The more predictable a high water event is, the higher the fraction of evacuation will be.

Because the SSM tends to underestimate the monetary values of damages not related to people and does not take into account certain damages, a calibration factor of 1.5 is used to enlarge the output to a more realistic value.

Cost-Benefit Analysis Province of Limburg

According to Frank Heijens from the Limburg Waterboard, there is no CBA for safety measures in Valkenburg. Safety measures and their cost to benefit ratio are usually assessed using a simple 'back of the envelope' calculation. The outcome of these calculations showed that extra safety measures are not cost-efficient. However, this might change after the immense damages of the 2021 flood.

Nevertheless, the waterboard has developed a CBA tool in the past. Until now, they have unfortunately not used it. The tool uses the *Waterschadeschatter* (WSS), translated *Water damage estimator*, to estimate the damages for a flood, and thus the potential benefits of a certain safety measure. These benefits include the following aspects (HKV, 2016):

- Direct damages
    * Damages on farmland and crops
    * Damages on infrastructure and utilities
    * Damages on real estate and movable assets
- Indirect damages
    * Evacuation and accommodating costs
    * Suspension of business activities
    * Loss of travel time

Direct damages are the result of direct contact with water, while indirect damages are a consequence of these direct damages. Indirect damages other than the ones mentioned, are not taken into account in this tool.

As can clearly be seen, the SSM takes into account damages related to people, while the WSS does not. Frank heijens stated that they are currently not taking into account any victims (as is confirmed by the WSS tool manual) because it is assumed that a flood resulting from the regional water system is not severe enough. However, the 2021 flood showed that victims might be plausible for future floods. Therefore, this is a limitation of the WSS tool which might result in an underestimation of the total benefits of a safety measure. Furthermore, the SSM takes into account the loss of revenue for businesses outside the flooded area, while the WSS does not.

Unfortunately, we cannot directly answer the question 'why the general safety standard is not achievable', resulting from section 3.2 because there is no CBA for the safety measures in Valkenburg. However, with the given limitations of the WSS tool, we would advise to integrate the SSM in the CBA instead of the WSS tool. Another solution would be to add damages related to people to the WSS tool. This is however, only possible if Deltares agrees. When using the SSM instead of the WSS tool in the CBA, benefits might increase which could lead to cost efficient measures (which would be not cost efficient according to the WSS tool). This finding could lead to a new perception where greater safety levels are achievable in Valkenburg, which could lead to the obligation to implement safety measures to reach that standard.

3.2.2 Individual and societal risk

The CBA does not fully determine the safety standards for areas vulnerable to flooding. In the past, the Dutch government has decided to set a certain safety level against flooding. For primary flood defences, there are laws stating that the standard protection level for each individual cannot be lower than 10^{-5} per year. This means that a person staying at any place behind a primary flood defence for 1 year long, has a maximum probability of dying from floods of 10^{-5} per year (Stowa, 2019).

Additionally, multiple casualties at once is considered less acceptable than the same amount of casualties in multiple events. For this reason the societal risk is defined in the Dutch law stating that the FN-curve (probability-casualties curve) should be below the limit line defined per area as can be seen in figure 3.4. The limit line, however, depends on certain undefined factors like the risk aversion index and policy factor to account for voluntariness of the exposure. Especially the first factor is a political choice.

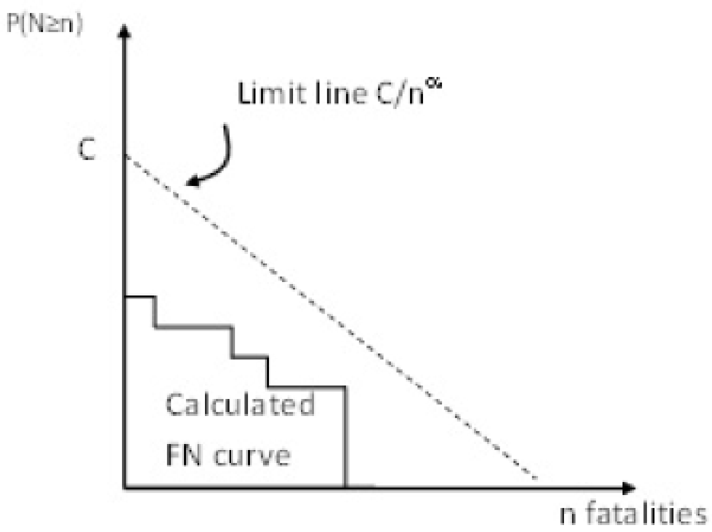


Figure 3.4: Societal Risk limit line (Jonkman et al., 2021).

This regulation only applies to locations behind the primary flood defences. This is because it is assumed that flooding behind regional flood defences will only result into nuisance and no actual danger to citizens. However, when considering the river basin of the Geul, this assumption is not valid. Therefore, it can be argued that the regulation for primary flood defences should also be valid at this location as this regulation should provide a minimum safety standard for each individual, irrelevant of their location within The Netherlands.

The probability of a certain flood, the amount of victims and the amount of inhabitants of a certain area together determine the individual risk in a certain area. A severe flood of the river Geul with a probability of $1/100$ year^{-1} can result 1-5 fatalities (Stuurgroep water, 2018). The total amount of inhabitants next to the river Geul can be seen in table 3.2. This gives an individual risk for each individual in the Geul area of $\frac{1}{648{,}740}$ which is considerably smaller than the required individual risk of $\frac{1}{100{,}000}$ and therefore satisfies this criterion when making the conservative assumption of 5 fatalities.

Also the probability of dying in a $\frac{1}{1{,}000}$ flood and lower probabilities should be added in order to create a complete picture. However, accurate modelling of these floods are required as the consequences for these events are still unknown. The modelling is, however, out of the scope of this research.

Table 3.2: Amount of inhabitants located next to river Geul (CBS, 2021)

Town	Inhabitants
Mechelen	1,791
Gulpen	3,950
Wijlre	2,450
Schoonbron	205
Schin op de Geul	695
Oud-Valkenburg	115
Valkenburg aan de Geul	10,500
Geulhem	60
Meerssen	7,441
Bunde	5,230
Total	32,437

The probability mentioned above was based on the entire populations living in municipalities along the river Geul. However, a fraction of the entire population living close to the Geul is actually vulnerable to flooding and therefore exposed to the risk of dying in a flood. So when calculating the risk considering only the people located in the flooded area, the probability of dying will be considerably higher than for the complete villages. In order to have a chance of $\frac{1}{100{,}000}$, 15.4% of the surface of the villages and towns has to be flooded. When assessing the inundation maps for a return period of 100 years, many of these places have a lower percentage of the urban area that is inundated. Therefore, it is arguable whether the individual risk is low enough.

In order to calculate the societal risk, factor C of the limit line has to be obtained. This can be done with the following formula developed by TAW working group 10 'probabilistic methods', 1985, and Vrijling et al., 1995:

$$C = (\frac{\beta * 100}{k * \sqrt{N_A}})^2 \tag{3.1}$$

in which β is the political factor accounting for voluntariness of exposure and is assumed to be 0.01. k is the risk aversion index and a common used value in The Netherlands is $k = 3$ (Jonkman et al., 2021). Since the river Geul flows through the entire area, the number of areas assumed is 1. When assessing the extreme scenario of the $1/100$ flood and conservatively assuming 5 fatalities, the following has to be true:

$$1 - F_N(n) \leq C/n^{\alpha} \tag{3.2}$$

α indicates the steepness of the limit line. According to the approach of TAW working group 10 'probabilistic methods', 1985, and Vrijling et al., 1995, α should have a value of 2 for the defined function of C. This means that 10 times more casualties should have a chance of occurring which is 100 times lower. Filling in the numbers stated above gives:

$$\frac{1}{100} \leq \frac{1}{45}$$

which is true and therefore, the societal risk is respected which gives no grounds for altering the flood safety level of Valkenburg when assessing the individual and societal risk when considering the full population of all cities and villages located next to river Geul and assessing a 1:100-flood. However, the risk for people in the area vulnerable flooding is considerably higher and is not in line with the regulation on maximum individual risks. Also the probabilities of more extreme circumstances should be included by including models.

3.2.3 The preference of the locals

The ALARP principle, CBA, and individual/societal risk are methods to quantitatively determine the appropriate safety level based on chance, benefits, and costs. These are methods that are implemented by governments or municipalities. However, they hardly take into account the emotional damages and the effect that it has on the well-being of the locals. To include the preference of the people affected by the safety measured as addressed in Jones-Lee and Aven, 2011, we performed an interview with citizens of Valkenburg. As stated in the methodology, this survey was conducted in person in the city centre of Valkenburg.

A total of 17 questions were asked (see Appendix **??**), some open and some closed, meant to both quantify certain aspects as well as to sketch a broad view of their mentality. The survey was deliberately kept very 'open' to make it feel more like a conversation. The idea behind this was to get people to talk openly and to get more information besides just the answers to our questions.

The first couple of questions were open questions meant to get people to talk about themselves and their experiences during the flood. Next, people were asked about their feeling of safety, both before and after the flood. These questions were meant to see if people were aware of the danger of flooding in Valkenburg before the flood happened. It can also serve as an indication if something has to change to get people to feel safer again. Next, a quantification of people's resilience to experience a flood was asked for. After explaining the current safety level of Valkenburg, they were asked to give their opinion on it to see if they felt it was acceptable or not.

The next part of the survey was about the participants' willingness to pay. They were asked how they would weigh an increase of safety level against a tax increase. How much are they willing to pay extra to see a doubling of the safety level? Similarly, they were asked to consider the aesthetics of the canals and the city. This was done by asking if they would care about raising the quay walls to increase safety. The last part of the survey consisted of open questions to get people's closing thoughts on the matter and see what creative solutions the locals themselves had already come up with. What measures would they like to see, and which measures are they against? What other aspects should play a role in defining the safety level, besides costs and safety?

The outcome of the survey was as follows: a significant portion of the surveyed were people of old age, with few business owners. The experiences during the flood varied per person with one similarity: the events were perceived having a huge impact, both physically and mentally. The amount of damage was generally low with one outlier of 115.000 euros to a house. Some people with low damage of their own still felt sentimental with regards to their fellow local residents, which indicates an event like this could also affect people indirectly. Almost all of these people also indicated that they wanted to pay extra to contribute to a higher safety level in Valkenburg.

The sense of safety before the flood was exceptionally high, with little awareness of the risks of a possible flood. As expected, the sense of safety decreased after the flood, but not to a sense of total insecurity. With respect to the safety level of Valkenburg and neighbouring urban areas, people were quite oblivious.

The general opinion towards the acceptable number of floods during a lifetime was that this number should be (close to) zero. This makes sense because there was no downside enclosed in the question. The next question added the consideration of costs and gave a more nuanced point of view. First a general dilemma was provided where low costs + low safety level was weighed against high costs + high safety level. The general opinion was in favor of high safety accompanied with high costs. Generally, there were two types of answers to these questions. One answer was that they wanted to pay whatever it costs to reach a higher safety level. One retired woman even stated that she would even start working again if this would be necessary. The other prevalent answer was that people did not want to pay anything at all. Generally, these people argued that they do not mind paying more but that they do not trust the local government with their money. Then, a quantitative question was given. To double the safety level of Valkenburg, people were willing to triple their water board tax. Fifty percent of people chose this option, compared to twenty percent for doubling the tax and even thirty percent for paying nothing extra.

The last part of the survey was about aesthetics and possible measures. Raising the quay walls, providing more protection but a less aesthetic view, had the upper hand, with an average increase of about 1 meter in height as maximum. Most people stated that they did not care about the aesthetics as long as they were better protected from floods, but usually the people working in tourism related businesses were a bit more in favor of preserving aesthetics wherever possible.

Lastly, possible solutions suggested by the citizens included: increasing the amount of green area in the centre, using water buffers and the existing idea of a water tunnel underneath Valkenburg. In these last questions we found that most people had already thought about possible measures, both for their own houses as well as for the entire village, and that most people were well informed on what measures could be possible and realistic.

3.3 The current physical flood defence system

To come up with possible safety measures to increase the safety level, one first has to analyse the current physical flood defence system present.

The Geul enters Valkenburg and then is split into two canals that flow through the historic city center. A weir divides the flow between the two canals, which can be seen in figure 3.5. The canals then rejoin at the other side of the city center. A large number of bridges span over these two canals, of which some could limit the discharge capacity of the canal by reducing the area of the canal under the bridge. There also is a watermill blocking the canal severely. Throughout the city there are quay walls, but at several places these quay walls show gaps, e.g. for balconies over the water. Some quay walls and bridges show severe damage and one of the larger bridges over the canal even collapsed. Furthermore, there are large rocks and vegetation on the canal bed that limit the flow. Besides the flood defence systems in the city centre, water buffers are present upstream which can hold a large amount of precipitation.

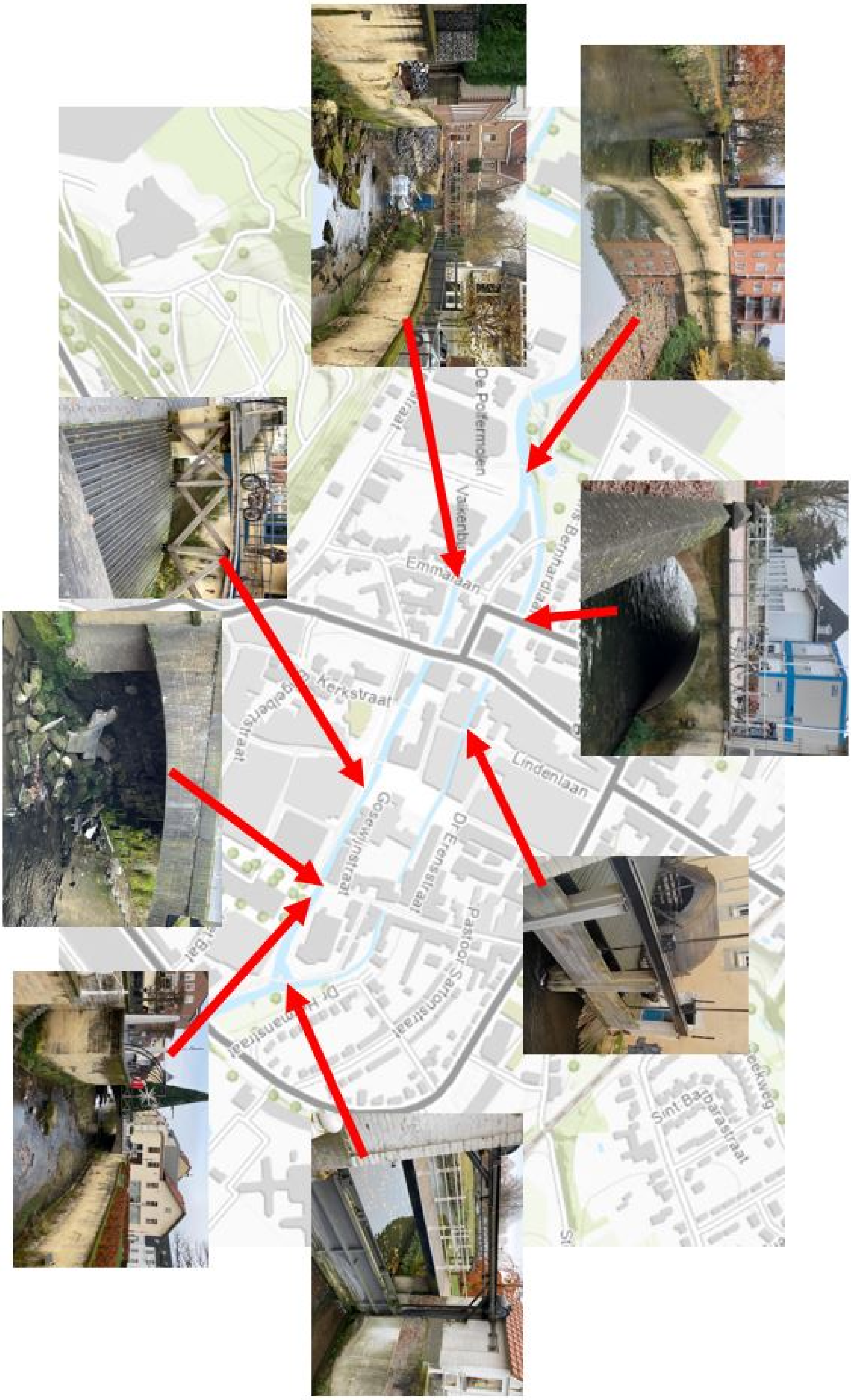


Figure 3.5: The current state of the flood defences of Valkenburg

3.3.1 Discharges and corresponding damages

If new measures should be taken and the flood defence system of Valkenburg will be improved, it is necessary to find the discharge that belongs to a certain safety level. In order to obtain a return period for certain discharges, the *Generalised Extreme Value-method* (GEV) is applied. The highest values of a certain time-step are selected. A time-step of a year is chosen and the data from 1970 to now is used to obtain these yearly maximums. Afterwards, these values are used to fit into a Gumbel-distribution. This distribution consists of the following parameters:

- Shape parameter: quantifies the heaviness of the tail
- Location parameter: locates the peak of the distribution
- Scale parameter: quantifies the spread of the extremes.

The lack of reliable discharge data is however a problem for Valkenburg. There is data available for the water heights in Valkenburg but there is no discharge data. Since the flood waves are by definition non-steady, it is extremely challenging to accurately estimate the discharges in Valkenburg. Therefore, the choice is made to use data of discharges at different locations. Meerssen has historical data of discharges and is located just downstream of Valkenburg and therefore would be a suitable option. However, the available data of discharges in Meerssen has only been measured since 2012 and therefore would only give 10 extremes. This available amount of data is not sufficient to perform a GEV. Therefore, the data of Hommerich is used which has been recorded since 1970. The catchment area at Hommerich is however considerably smaller than the catchment area which results in the discharge into Valkenburg. Therefore, the discharge has to be multiplied by a factor after determining the discharges of certain return periods at Hommerich. One additional problem is that during the floods in the summer of 2021, the measuring equipment was severely damaged in such a way the measurements are not accurate anymore. However, the floods were so extreme and therefore greatly impacting the GEV-analysis and return period, which makes it important to include the event. Therefore, the discharge is estimated with the formula for equilibrium depth calibrated with previously high water data resulting into a discharge of 56.9 m^3/s. When applying the GEV-analysis, the *probability density function* (pdf) of Hommerich is obtained and can be seen in figure 3.6.

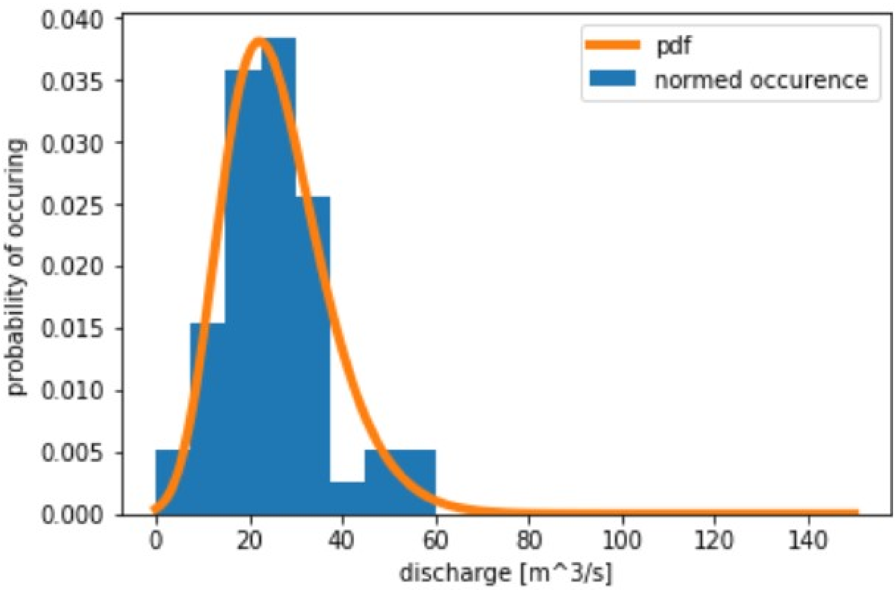


Figure 3.6: The obtained pdf of discharges in Hommerich

Also, the *cumulative density function* (cdf) is obtained and can be seen in figure 3.7. The cdf can be intersected at certain cumulative probabilities in order to match return periods of certain discharges. In this way, a cumulative probability of 24/25 corresponds to a discharge with a return period of 25 years. This leads to discharges shown in table 3.3. The python code made to perform this analysis is shown in **??**.

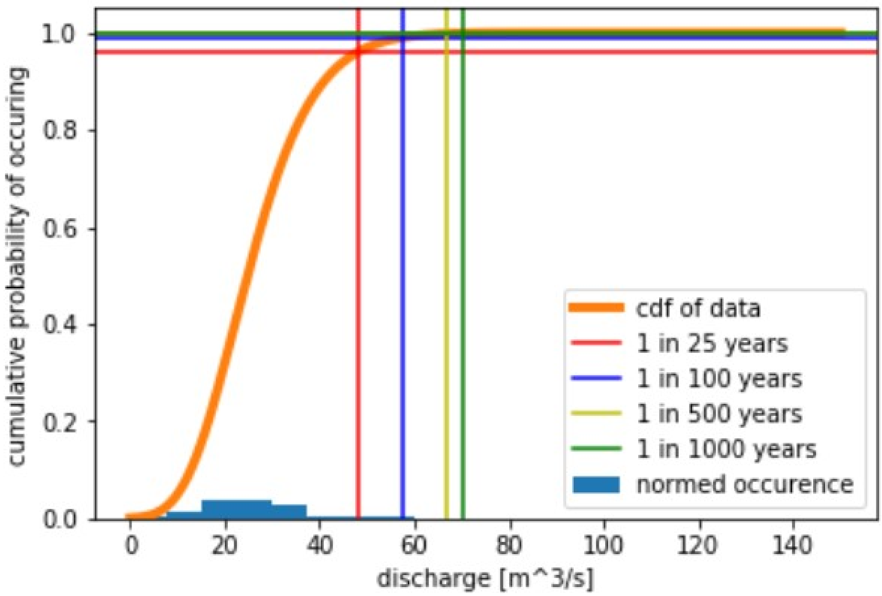


Figure 3.7: The obtained cdf of discharges in Hommerich with certain return periods

Table 3.3: Discharges with certain return periods for Hommerich

Return period	**Discharge [m^3/s]**
25 years	48.0
100 years	57.4
500 years	66.8
1000 years	70.4

Finally, the discharges of Hommerich should be multiplied by a factor accounting for the bigger catchment area Valkenburg has compared to Hommerich. The water level in Valkenburg is known for the same period data of discharges in Hommerich is available. For the years, the yearly highest water level in Valkenburg coincides with the yearly highest peak discharge in Hommerich, the discharge is estimated by assuming assuming uniform, non-steady flow as there are little to no obstructions just before the measuring location in Valkenburg. Furthermore, high water waves generally have relatively long peaks. A factor can now be obtained when dividing the estimated discharges of Valkenburg by the measured discharges in Hommerich for the years the peak discharge and peak water level coincide. The mean of these different factors is 1.48 and is comparable to the factor for the difference in catchment area for the two locations. The return periods of Valkenburg can now easily be calculated by multiplying this factor to the discharges of Hommerich. These are shown in table 3.4.

Table 3.4: Discharges with certain return periods for Valkenburg

Return period	**Discharge [m^3/s]**
25 years	71.2
100 years	85.1
500 years	98.9
1000 years	104.3

The influence of climate change

The water safety level norms are based on data of the past. Therefore, the future climate change is not accounted for and can result into a difference between the designed safety level and the actual safety level of the future. Although it is impossible to observe the compounding effect of climate change in data from the past, an increase in the intensity of the extremes can already be observed, as can be seen in figure 3.8. It is highly arguable to assume a linear regression but this does show the impact climate change might already have had (since it can also be a matter of coincidence).

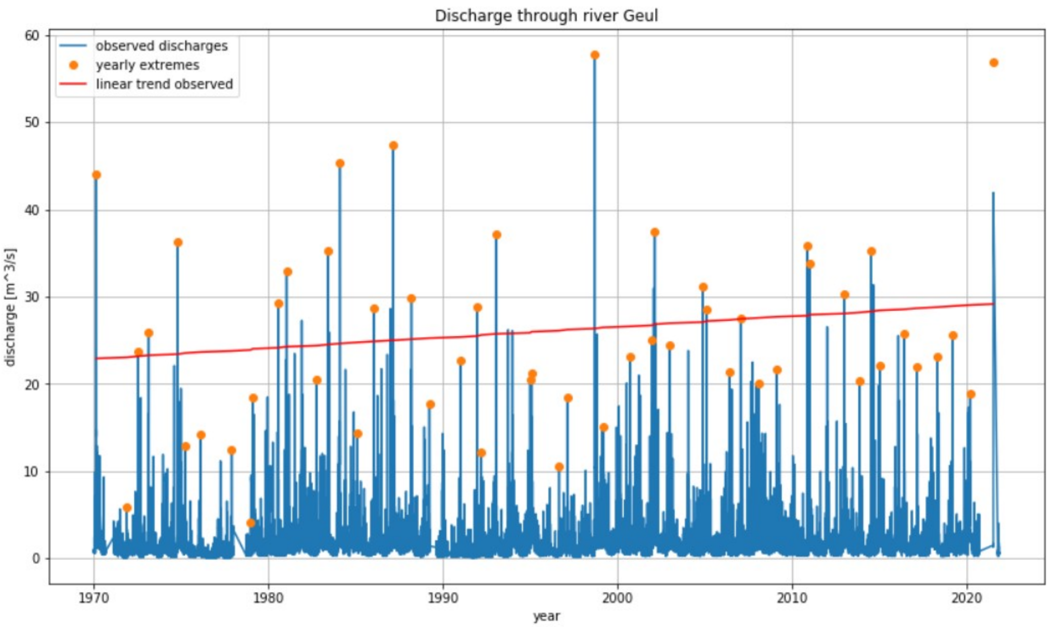


Figure 3.8: Linear regression for the discharges in Hommerich

The discharges in 2050 would increase by a factor of 1.12 compared to the discharges of 2021 when extrapolating the red line to 2050. However, as previously stated, it is arguable to assume linear regression as the intensity of the rainfall is related to the temperature for which the predictions are non-linear. As a result of the rising temperature, the air can hold more moisture; 7% more moisture can be kept in the air for every degree Celsius according to the Clausius-Clapeyron relation (KNMI, 2011). This relation represents the actual humidity when enough water is available which is the case in The Netherlands. However, a 7% increase of humidity cannot directly be converted to a 7% increase of rainfall-extremes which makes linear regression even less appropriate. The link between humidity and increase of rainfall extremes is still highly uncertain and is estimated with a range between a 2% and 14%-increase per degree Celsius (KNMI, n.d.). This high uncertainty is due to the fact that for extremely humid and warm conditions, these clouds can develop considerably faster. This generally happens in the summer months. A 10%-increase of intensity by approximation leads to doubling of the chances of exceedance of a certain threshold value. For extreme events, chances increase even further. When assuming a 14%-increase per degrees Celsius and the KNMI'14 climate scenarios predict an increase of temperature between 1.2 and 3.6 degrees Celsius compared to now and therefore, the extremes in intensity can increase by 17% to 60%. Other researches even have a bandwidth of an increase of +20% to +800% Expertisenetwerk waterveiligheid (ENW), 2021. Therefore, it can be concluded that the impact of the changing climate is highly uncertain and so, **the impact of the changing climate is not taken into account for the estimates of the impact of measures**. The impact of climate change on the intensity of rainfall is more thoroughly researched by Athanasios Tsiokanos (MSc student TU Delft) who is currently doing his master thesis on this topic.

Damages

Now the discharges of the most common return periods are known, we can start coupling damages to them. A side note has to be added here that these are not very accurate estimates, since a lot of aspects are still unknown, i.e. what return period the discharge of the 2021 flood had, what the exact damages of this flood were, and what the damages would be for other floods with different return periods.

As a departure point, we need to find the maximum discharge that is able to occur without causing any damages. Since the safety level of Valkenburg is set at a return period of 25 years, we assume no damages at this specific discharge of 71.2 m^3/s. When discharges exceed this value, nuisance and floods occur, which induce damages. To find out the monetary value of damages corresponding to different discharges, a tool like the WSS or SSM could be used, as discussed before. However, sine the use of these models does not correspond with the scope of this research, assumptions have to be made. Researches such as Velasco et al., 2016, Mcgrath et al., 2019, and Wu and Guo, 2021, show that there is no one correct correlation between water level depth (and thus discharges) and damages. This correlation depends on multiple factors, e.g. the kind of buildings and the geographical features in the flooded area. In this research, a linear correlation is assumed between discharge and damages. Other correlations that could be argued about are step wise or exponential correlations.

Since the exact discharge and damages from the 2021 flood are not known, we assume that a discharge corresponding with a return period of 1000 years occurred (104.4 m^3/s), which induced €400 million in damages. This results in a discharge-damage curve which can be seen in figure 3.9. The damages that occur at a discharge greater than 104.4 m^3/s are also unknown. They could be limited up until €400 million, continue to rise according to the same linear correlation slope, or rise even more progressive. For this reason, this region in figure 3.9 is indicated with a dotted line.

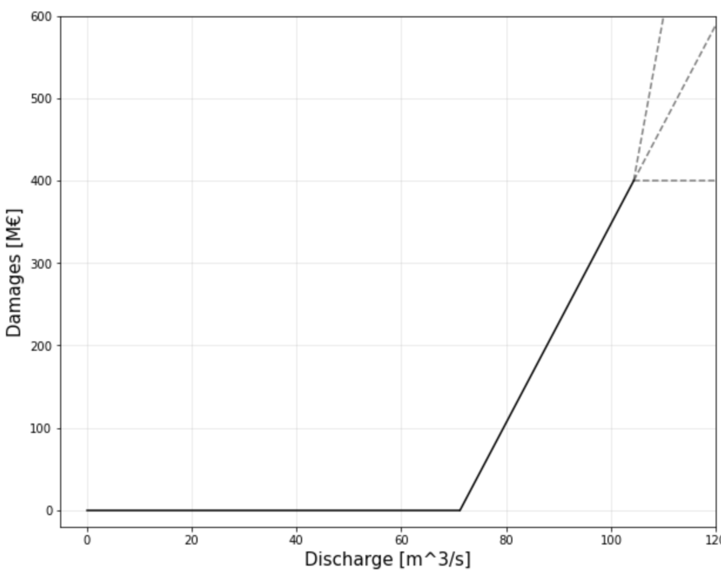


Figure 3.9: Discharge-damage curve

4. Conceptual design and solutions

This chapter will focus on the development, elaboration and evaluation of different conceptual design alternatives to improve the safety level of Valkenburg. On top of that, increasing the social acceptance for flood risk in Valkenburg is discussed.

4.1 Conceptual design of hydraulic solutions

This section will present multiple solutions to tackle the problem of flooding. Solutions will vary between high and low impact and will all be elaborated.

4.1.1 (Re)Designing the bridges

In section 3.3 we have seen that there are some old masonry bridges in the city center of Valkenburg. These bridges limit the discharge capacity of the Geul and can cause a backwater curve. An option to increase the discharge capacity of the Geul is to (re)design the bridges over the Geul so that they interfere less with the flow. As a consequence of the flood, one of the bridges over the Geul collapsed, as can be seen in figure 3.5. This failure asks for a redesign of the bridge, with the future kept in mind. This new design has to provide the same functions as the old bridge in terms of traffic, but also has to be resilient in case of another flood. On top of that, the new bridge design has to be as such that a new event of high water must not be worsened by the presence of the bridge.

Essentially, there are 2 different design options: a bridge that can move in case of high water or a flat bridge that interferes as little as possible. In the coming sections, discharge capacities of the different design options are analysed, loads on the bridges are identified and a preliminary design for the collapsed bridge is given.

Influence of bridge design on discharges

In order to assess the difference a flat bridge can make, the maximum discharge for the arch bridges is computed and compared to the discharge with flat bridges. There are several arch bridges in Valkenburg and the estimated dimension of the bridges are 3.5m x 2.3m in the shape of an oval, as can be seen in figure 4.1. The depth of the river is estimated to be 2.7 meters. This bridge is located in the northern branch of Valkenburg.

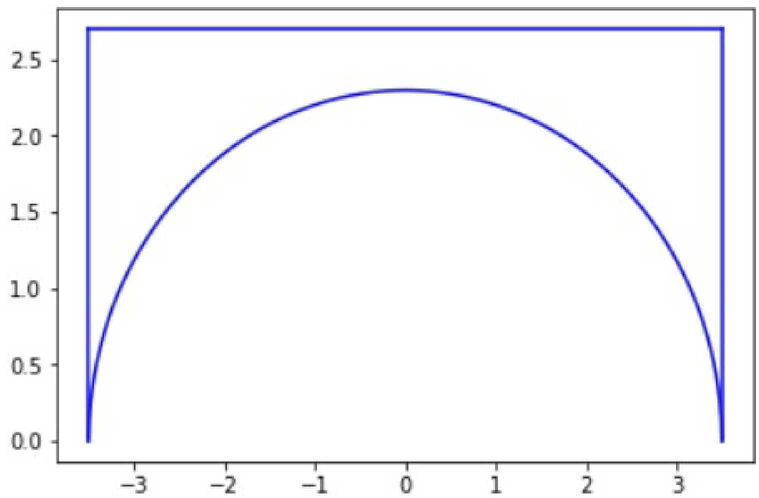


Figure 4.1: The front view arch bridge

The bridges can be considered culverts. The maximum discharging capacity of a culvert is reached just before reaching the top of the culvert as can be seen in 4.2.

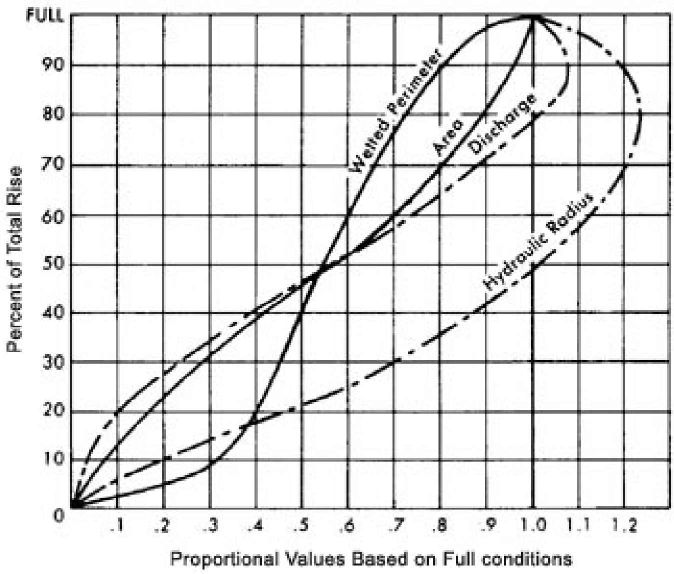


Figure 4.2: The maximum capacity of culverts Highway Task Force, 1970

Therefore, the maximum discharge capacity can be calculated when the culvert is not completely full and can be considered open channel flow. Now, the discharge can easily be obtained by the Strickler-Manning equation:

$$Q = \frac{1}{n} A R_h^{\frac{2}{3}} s_0^{\frac{1}{2}} \tag{4.1}$$

With:

$$R_h = \frac{A}{P}$$

In this function, the A represents the flow area, R_h represents the hydraulic radius, P represents the perimeter, s_o represents the bed slope and n represents the Manning's coefficient. Since the bridge is almost completely full, the area is by approximation equal to the area below the bridge which can be obtained by integration below the arch as shown in 4.1. Also the perimeter can be obtained with this function. The bed slope is equal to 2.3e-3 and the Manning's coefficient is assumed to be 0.014 obtained from Elger et al., 2014a. The friction coefficient is extremely sensitive and it would be useful to calibrate it for this situation. However, that is not possible since no discharge data is available for Valkenburg and the situation at the measuring stations in Meerssen and Hommerich are not comparable as these channels are unlined while the channels in Valkenburg are lined. The friction losses due to contraction of inflow are assumed to be negligible. This results into a discharge of 45.7 m^3/s for the northern branch in Valkenburg. When assuming a 70/30%-ratio for discharge for the northern and southern branch based on the width of entrance, the full discharge through Valkenburg is 65.3 m^3/s, corresponding to a return period of just of approximately 15 years. This is a bit lower than the actual capacity that is Valkenburg is designed for. That is possible as the bridges in the southern branch contain arches which do not start at the bottom. Therefore, the discharging capacity of the southern branch is likely to be a bit higher than assumed. The python-script to calculate these discharges can be found in **??**.

The same can be done for the flat bridges. As a first estimate, the thickness of the bridge is assumed to be the span divided by 20. This results into a thickness of 35 cm for the same canal dimensions. An additional 15 centimeters is assumed for extra layers like asphalt. Therefore, the flat bridge is assumed to be 50 centimeter thick and is shown in figure 4.3.

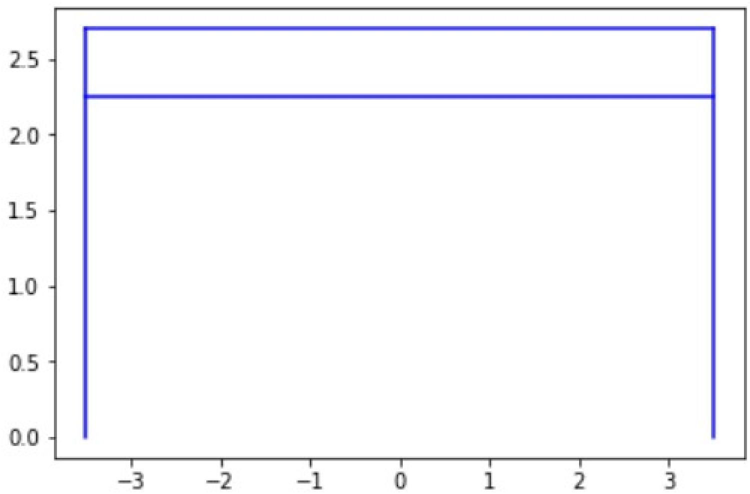


Figure 4.3: The front view of the flat bridge.

The discharge capacity can again be described with equation 4.1. The area is simply the width of the bridge time the maximum water depth without reaching the bottom of the bridge. The roughness coefficient is now considerably higher since the river bed relatively has more influence on the roughness coefficient as it is a larger percentage of the total wet perimeter (since the perimeter of the flat bridge is smaller than for the arch bridge). Therefore, a value of 0.16 is assumed. This results into a total discharge of 80.1 m^3/s, corresponding to a return period of approximately 60 years.

Also liftable bridges can be used. The maximum water depth is now increased to ground level. The total discharge will be 107.2 m^3/s corresponding to a return period of over close to 1500 years.

When combining the liftable bridge and filling up the holes in the quay wall, an additional meter of conveying capacity is included. This results into an infinitely high return period. This is probably due to the fact that the flow the short period of available data in which extreme discharges of this magnitude are not included. Also, the discharge is highly sensitive to a change in the friction factor which can lead to an overestimation of the discharge capacity. This is described in more detail in the discussion.

Designing a flat bridge

In the previous section, the effect of different bridge designs are discussed. As expected, the alternative of a flat bridge is found to increase the discharge with respect to an arched bridge. In the coming section, this information is used to make a conceptual redesign of the collapsed bridge using a flat shape. The outcome of this will provide recommendations for such a redesign. This will be done by considering the requirements and loads, and using this to make a basic structural analysis.

The requirements of the bridge are summed up below, with the layout visualized in figure 4.4.

- Span: 10 m
- Width: 12 m
- Layout:
  - 2 sidewalks
  - 2-way driving lane
  - 1 parallel parking lane
- Load:
  - Parked vehicles
  - Pedestrian
  - Light traffic
  - Wind loads
  - Snow loads
  - Water load
  - Debris impact load
- Working life: 100 years

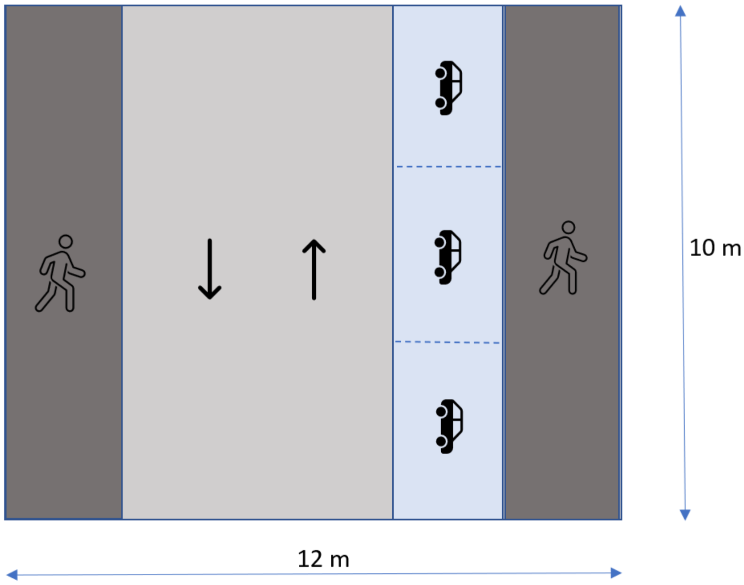


Figure 4.4: Top view layout bridge

Loads

To design the bridge, numerous loads need to be taken into account. This section focuses on identifying all relevant types of loads and providing (basic) numerical assumptions. Besides the usual loads for designing a bridge, like self-weight, traffic loads, impact loads and wind loads, this specific bridge should also be able to cope with extra loads in case of a flood. The flood actions that need to be taken into account are: hydrostatic actions, hydrodynamic actions, Buoyancy action, and impact by debris (Kelman and Spence, 2004).

If all dead and live loads are obtained, the Ultimate Limit State (ULS) and Serviceability Limit State (SLS) can be checked. For ULS, the total design load is calculated with the formula:

$$\gamma_G \cdot G_k + \gamma_{Q;1} \cdot Q_{1;k} + \sum(\gamma_{Q;i} \cdot \psi_{0;i} \cdot Q_{i;k}) \tag{4.2}$$

Here the γ's represent partial factors, Q represents live load, G represents dead load and ψ is a combination factor.

The formula for determining the design load in the SLS is as follows:

$$G_k + Q_{1;k} + \sum(\psi_{0;i} \cdot Q_{i;k}) \tag{4.3}$$

The limit states are checked by considering all possible load combinations and testing the most unfavourable one(s). In short, SLS is used for checking the bridge with respect to deformations, while the ULS is used for strength verifications. To do this, first the numerous loads have to be defined.

Self-weight

First, materials and dimensions have to be chosen. As stated before, the thickness of the structure has to be as low as possible as to not obstruct the water. With a span of 10 meters and no exceptionally high loads, a suitable solution is prefab hollow core slabs. A width of 1.2 meter and height of 0.32 meter are a possible cross section. This would mean 10 hollow core slabs next to each other with spans of 10 meter each. A look in the quick reference (Delft

University of Technology, 2016) provides a maximum applied load of 18 kN/m^2 for a span of 10 meters.

Figure 4.5: Bridge deck composed of hollow core slabs

The slab has a self-weight of 4.43 kN/m^2. A topping would have to be added of in-situ concrete to activate diaphragm action between the slabs. An estimation of this layer is 10 cm in height. With a density of 2400 kg/m^3, this gives a load of $2400 \cdot 9.81 \cdot 0.1 = 2.35$ kN/m^2. Furthermore, asphalt and/or brickwork needs to be accounted for. When considering asphalt, the self-weight is around 2300 kg/m^3 and a thickness of around 10 centimeter is assumed. This gives a dead load of $2300 \cdot 9.81 \cdot 0.1 = 2.256$ kN/m^2. This makes the total self-weight 9.04 kN/m^2. A schematic image is given in figure 4.5 above.

Traffic loads

The traffic loads that need to be considered can be taken from the Eurocode (EN 1992-2), which provides design criteria for road bridges and footbridges with the use of load models for different uses of the bridge.

Firstly, the type of bridge and type of load need to be determined. The bridge is a standard road and pedestrian bridge for which load model 1 is used (Eurocode, 2003). The effective width of the carriage way is estimated at 5.20 meters, resulting in 1 notional lane of 3 meters wide as seen in figure 4.6.

Carriageway width w	Number of notional lanes	Width of a notional lane w_l	Width of the remaining area
$w < 5{,}4\,m$	$n_l = 1$	3 m	$w - 3m$
$5{,}4m \le w < 6m$	$n_l = 2$	$\frac{w}{2}$	0
$6m \le w$	$n_l = Int\left(\frac{w}{3}\right)$	3 m	$w - 3 \times n_l$
NOTE For example, for a carriageway width equal to 11m, $n_l = Int\left(\frac{w}{3}\right) = 3$, and the width of the remaining area is 11 - 3×3 = 2m.			

Figure 4.6: Carriageway width and corresponding number of notional lanes (Eurocode, 2003)

This means the carriageway is modeled as 1 notional lane of 3 meters wide and remaining area of 2.2 meters wide. The corresponding load for the notional lane is 9 kN/m^2 distributed load and two axle loads of 300 kN. The remaining area is loaded with only a distributed load of 2.5 kN/m^2. Besides the traffic load on the driving lanes, a parking lane and two pedestrian lanes are also present. For traffic loads on bridges, the Eurocode doesn't provide loads for parked vehicles. Eurocode 1991-1-1: Actions of structures does provide this, but only for buildings such as garages. These values are taken as a reference. This means a characteristic distributed load p_k of 5 kN/m^2 and a characteristic concentrated load Q_k of 40 kN. The load of pedestrian lanes is described in EN 1992-1: Traffic loads on bridges and is recommended at $q_k = 5$ kN/m^2. This would give the following total load distribution as seen in figure 4.7.

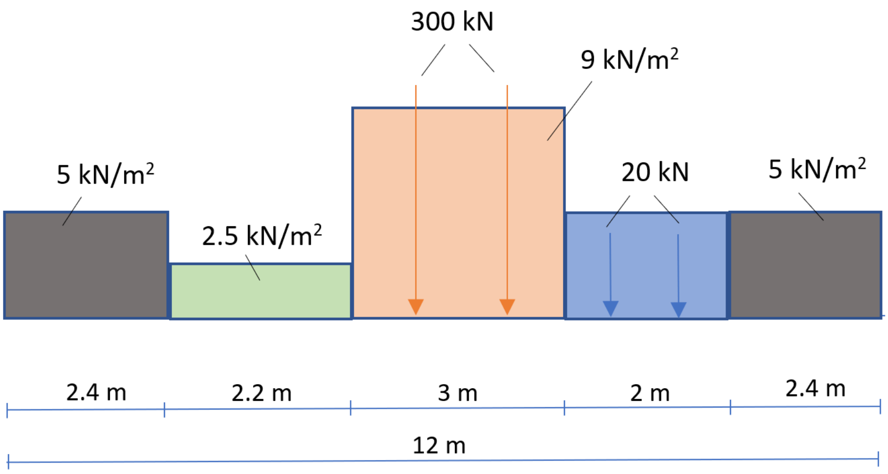


Figure 4.7: Total traffic loads

The most loaded beam will be evaluated in the end, so in this case that is one of the beams carrying the notional lane with 9 kN/m^2 and 2 x 300 kN. Due to the prefab beams and in-situ layer on top acting as a slab, both the distributed loads and concentrated loads will be transferred to multiple beams. For the distributed load this is negligible, but a point load will not be carried by just the beam directly underneath. So the assumption is that the heaviest loaded beam takes a full 9 kN/m^2, but not a 300 kN point load. The percentage of point load that will be taken up by one beam is hard to determine, so a rough estimation has to be made. The point load will be converted to the effective point load by assuming a load path as follows:

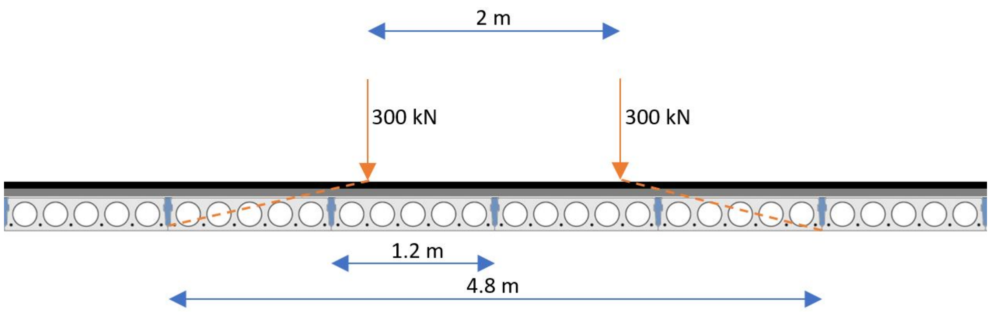


Figure 4.8: Load path point load

The effective load then becomes:

$$F_{eff} = \frac{2 \cdot 300}{4.8} \cdot 1.2 = 150 \text{ kN}$$

This effective point load can be converted to a distributed load by looking at which distributed load would give the same maximum bending moment. This is done by setting the maximum moment in longitudinal direction caused by a point load in the middle of a beam equal to the maximum moment caused by a distributed load. As follows:

$$\frac{1}{4}Fl = \frac{1}{8}q_{eq}l^2$$

In this way, a point load of 150 kN would give an equivalent distributed load of 30 kN/m. Dividing by the width of one beam gives an load of 25 kN/m^2. This means the total characteristic distributed load is $9 + 25 = 34$ kN/m^2.

Wind loads

The wind loads acting on the structure need to be determined for only the bridge deck, as the abutments are integrated in the soil and quay walls. Two directions need to be considered: horizontal and vertical. The load due to wind can be determined with the following formula:

$$F_w = C_sC_d \cdot C_f \cdot q_p(z_e) \cdot A_{ref} \tag{4.4}$$

Here C_sC_d is the structural factor, C_f is the force coefficient, $q_p(z_e)$ is the peak velocity pressure and A_{ref} is the reference area. The peak velocity pressure is a function of the height above ground and can be estimated depending the wind region and coastal/rural/urban distinction. This value of $q_p(z)$ can be taken from the quick reference (Delft University of Technology, 2016). With the bridge being around 2-3 meters above reference (water) level, the pressure is found to be 0.49 kN/m^2. C_sC_d can be taken as 1 and C_f differs for x and z direction. EN 1991-1-4 provides recommended values for the force coefficients and can be taken as $C_{f,x}$ = 1.3 and $C_{f,z}$ = ±0.9. A_{ref} will be clear when the dimensions of the bridge are chosen.

Snow loads

Because of the bridge being located in the Netherlands, snow loads have to be taken into account. The characteristic value of snow load in the Netherlands can be taken as 0.7 kN/m^2 on flat surfaces (Delft University of Technology, 2016).

Water loads

There are two main types of water loads, hydrostatic and hydrodynamic loads.
Hydrodystatic loads are loads related to the lateral pressure of water, see q_H in figure 4.9. The hydrostatic pressure on the side of the bridge can be calculated using the following formula:

$$P = \rho_w g b h \tag{4.5}$$

where h is the water depth. The resulting force due to hydrostatic loads can then be calculated by:

$$F_{hydrostatic} = \frac{1}{2}\rho_w g b h_{max}^2 \tag{4.6}$$

where h_{max} is the water depth at the underside of the bridge.

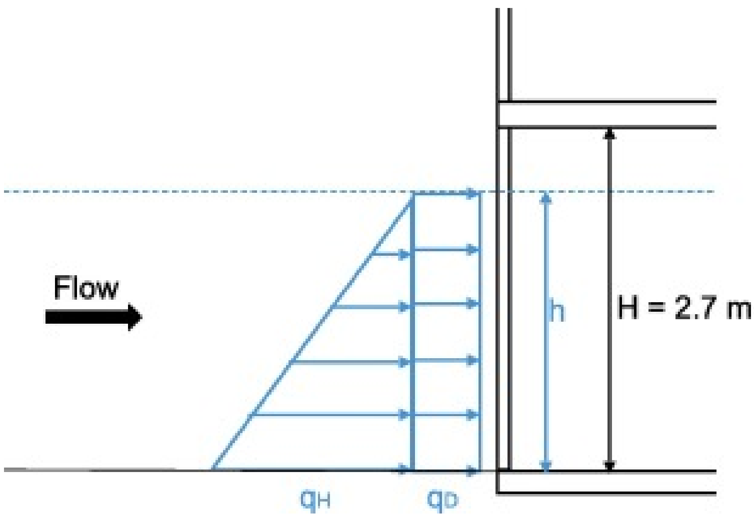


Figure 4.9: Water loads on a wall (Jansen et al., 2020)

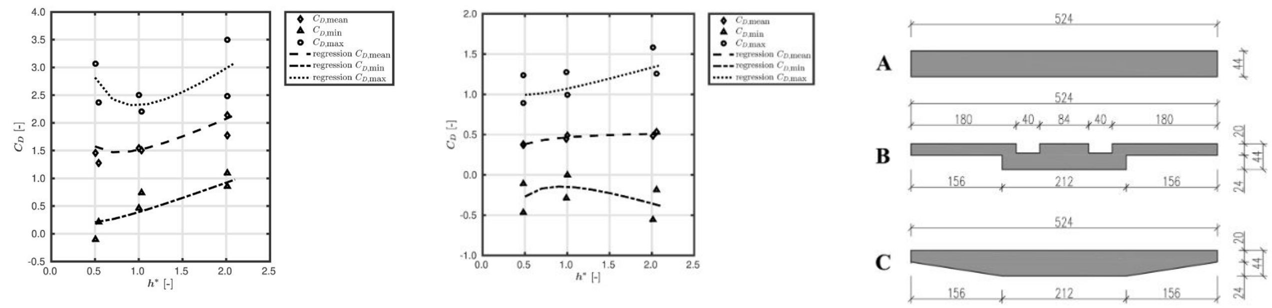
Another factor that has to be taken into account is hydrodynamic pressure. This is related to the water colliding with the bridge. The pressure is proportional to the velocity of the water squared. The hydrodynamic pressure can be calculated by:

$$P_{dynamic} = \rho_w C_D v^2 \tag{4.7}$$

where v is the velocity of the water and C_D is the drag coefficient. The Hydrodynamic force then becomes:

$$F_{dynamic} = \frac{1}{2}\rho_w C_D v^2 b h_{bridge} \tag{4.8}$$

The drag coefficient is dependant on the shape and the submergence of the bridge. In figure 4.10 the drag coefficient of a bridge with dimensions 524 by 44 mm can be found as a function of the relative submergence for turbulent and sub-critical flow. The relative submergence is defined as the water height measured from the underside of the bridge to the surface, divided by the height of the bridge. As can be seen in the figure, the mean drag coefficient is somewhere between 1.5 and 2.0. However, the values for the drag coefficient of similar shapes vary wildly between studies.



(a) Dependence of drag coefficient CD on the relative submergence of bridge deck A

(b) Dependence of drag coefficient CD on the relative submergence of bridge deck C

(c) The different bridge deck designs used in the study

Figure 4.10: Determination of the drag coefficient for different bridge designs (Drab et al., 2019)

The shape of the bridge deck can be optimized to significantly decrease the drag coefficient. Comparing bridge deck A and C in figure 4.10c, there is a very large decrease in C_D, from 1.5 to 0.5. This can be used in bridge designs to significantly decrease the horizontal load on the bridge.

Similarly, the flow of water over and under the bridge can create uplift or extra vertical load on the bridge. In figure 4.11 we see that the uplift coefficient on the bridge is highly dependant on the relative submergence, but less so on the shape of the bridge.

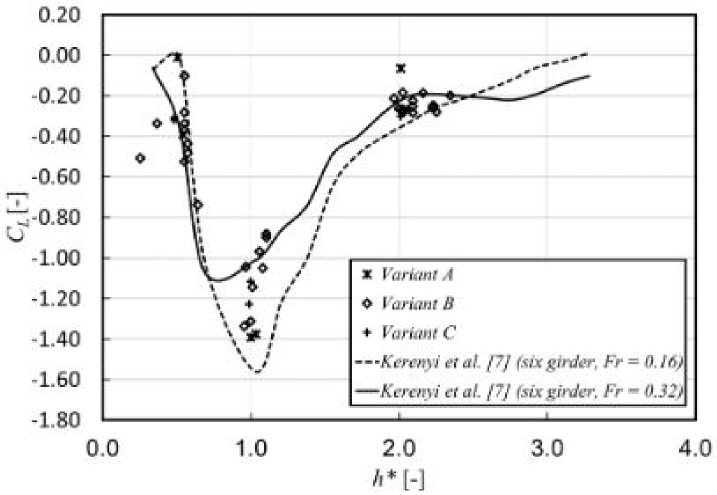


Figure 4.11: Lift coefficient on different bridge decks (Drab et al., 2019)

The uplift force acting on a bridge deck is calculated by:

$$F_{lift} = \frac{1}{2}\rho_w C_L v^2 bL \tag{4.9}$$

Where L is the length in flow direction and C_L is the lift coefficient, determined with figure 4.11 and the value of h* which is:

$$h^* = \frac{h_u - h_b}{s} \tag{4.10}$$

where h_u is the total height of water, h_b is the height of water up till the bottom of the bridge deck and s is the thickness of the bridge deck.

The static and dynamic water loads depend on the water level, flow velocity and coefficients. These parameters change in the different stages during a flood. Because of that, three stages are distinguished.

Stage 1: Water level up to bottom of bridge

If the flowing water level is up to the bottom of the bridge, the vertical pressure of the water is close to zero. The only load acting on it is the horizontal friction force due to flowing water. This force is negligible compared to the loads of further stages, so stage 1 is not governing.

Stage 2: Water level up to top of bridge

If the flowing water level is up to the top of the bridge, there is a static vertical pressure under the deck, a static horizontal pressure to the sides of the deck, a horizontal dynamic pressure to the sides of the deck and a dynamic lift pressure under the deck. The magnitude of the static forces are:

$$q_{ver,stat} = \rho_w g h = 1000 \cdot 9.81 \cdot 0.52 = 5.10\, kN/m^2$$

$$q_{hor,stat} = \tfrac{1}{2}\rho_w g h^2 = \tfrac{1}{2} \cdot 1000 \cdot 9.81 \cdot 0.52^2 = 1.33\, kN/m$$

This $q_{hor,stat}$ is a distributed line load at 1/3 of the height of the bridge deck.

For the dynamic forces, a flow velocity, drag coefficient and lift coefficient have to be determined. The flow velocity can be calculated with the simple formula:

$$v = Q/A \tag{4.11}$$

Data from the recent flood is used, with the discharge estimated at 100 m^3/s (Expertisenetwerk waterveiligheid (ENW), 2021). The area of the flowing water is estimated at 4 meters high by 7 meters wide. This gives a flow velocity of 3.6 m/s.

If we assume the bridge design to optimize the water discharge, option C is chosen as seen in figure 4.10c. The corresponding drag coefficient is 0.5. This gives a horizontal hydrodynamic distributed load along the beam of:

$$q_{hor,dyn} = \frac{1}{2}\rho_w C_D v^2 = \frac{1}{2} \cdot 1000 \cdot 0.5 \cdot 3.6^2 = 3.24\, kN/m^2$$

The lift coefficient is also calculated for bridge design C. The relative submergence h^* is 1 for this stage giving a C_L of around -1.2. This gives:

$$q_{ver,dyn} = \frac{1}{2}\rho_w C_L v^2 = \frac{1}{2} \cdot 1000 \cdot -1.2 \cdot 3.6^2 = -7.78kN\; kN/m^2$$

Stage 3: Water level 1 meter above top of bridge

If the flowing water level is 1 meter above the top of the bridge, there is a static vertical pressure under the deck, a static horizontal pressure to the sides of the deck, a horizontal dynamic pressure to the sides of the deck and a dynamic lift pressure under the deck. The static vertical water pressure can be calculated as a resultant pressure, subtracting the pressure on top of the deck from the pressure below the deck. The magnitude of the static forces become:

$$q_{ver,stat} = \rho_w g h = 1000 \cdot 9.81 \cdot (1 - 1.52) = 5.10\, kN/m^2 \text{ acting upward}$$

$$q_{hor,stat} = \frac{1}{2}\rho_w g h^2 + \rho_w g h^2 = \frac{1}{2} \cdot 1000 \cdot 9.81 \cdot (1.52 - 1)^2 + 1000 \cdot 9.81 \cdot 1^2 = 11.15\, kN/m$$

For simplicity, the distributed line load is converted to a distributed area load by dividing the load by the height of the deck. This is not entirely accurate but gives a good indication and makes the load much more simple to integrate in the total loads. This gives $q_{hor,stat} = \frac{11.15}{0.52} = 21.44\, kN/m^2$.

The magnitude of the dynamic forces change slightly with respect to stage 2. Where the flow velocity and drag coefficient remain the same, the lift coefficient changes because of a new relative submergence. The new relative submergence becomes:

$$h^* = \frac{1-0.52}{0.52} = 0.92$$

Giving a C_L of -1.1. This gives the following dynamic loads:

$$q_{hor,dyn} = \frac{1}{2}\rho_w C_D v^2 = \frac{1}{2} \cdot 1000 \cdot 0.5 \cdot 3.6^2 = 3.24\, kN/m^2$$

$$q_{ver,dyn} = \frac{1}{2}\rho_w C_L v^2 = \frac{1}{2} \cdot 1000 \cdot -1.1 \cdot 3.6^2 = -7.13\, kN/m^2 \text{ acting upward}$$

When looking at all stages, stage 3 seems to be governing and will be used moving forward.

Impact loads

An explanation of the collapse of the bridge could be the impact of debris floating on the water. While this is still speculation, a redesign should account for this type of loading for future purposes. This specific type of impact load is however not accounted for in the Eurocode. Only the impact of debris to piers is standardized in certain Eurocodes. For a new design, this impact load to piers will be used as a reference for impact loads to the lateral side of the superstructure which is the case in Valkenburg. The formula for determining the impact load for loads to piers is:

$$F_{dm} = \frac{1}{2}\rho_w C_D B_d h u^2 \tag{4.12}$$

Here ρ_w is the density of fluid, C_D is the drag coefficient, B_d is the cross stream width of debris, h is the water depth and u is the flow velocity. To get a numerical estimation of this force, the size of debris has to be estimated. The most reasonable type of debris is brickwork, estimated at a width of 20 cm. This gives the following impact force:

$$F_{dm} = \tfrac{1}{2} \cdot 1000 \cdot 0.5 \cdot 0.2 \cdot 5 \cdot 3.6^2 = 3.24\, kN$$

It has to be noted that this load is very insignificant in the total loading scheme. A sizeable load like a tree trunk would be more significant, but very unlikely.

Another form of impact loads is the impact of a wave on the bridge deck. While a wave will not be formed by the rainwater itself, it is possible that an event upstream can cause a wave, like a collapsing dam or bridge. Therefore, when designing a bridge, this might be taken into account. As it is unlikely that a wave will happen in Valkenburg, this will not be included in our calculations, but we will discuss it briefly for the sake of completeness. The impact force of a wave depends on the geometry of the bridge and canal, and the flow characteristics. Xu et al., 2021, experimentally found formula 4.13 for the force on a bridge due to a wave impact.

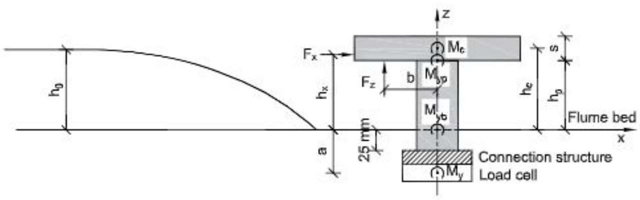


Figure 4.12: The layout of the experiment for determining the tsunami wave impact force (Xu et al., 2021)

$$\frac{F_{x,\max}}{\rho g h_0 A_{h2}} - \left\{ (0.7\,\mathrm{Fr}_b + 1) - 1.9\tfrac{h_p}{h_0}, h_p < h_0 0.87, h_p \geq h_0 \right. \tag{4.13}$$

Where h_0 is the bore depth (see figure 4.12), F_x/A_{h2} is the net pressure on the bridge deck in flow direction, h_p is the height of the bridge deck measured from the flume bed and F_{r_b} is the Froude number of the bore.

Non-physical loads

Besides the physical loads on a bridge, a bridge is also subjected to environmental effects, like carbonation in concrete or fungi in timber structures. In the event of a flood, the structure might come into contact with the water. In design codes, non- physical loads are taken into account with the use of durability classes, thickness of the concrete cover, and/or conversion factors. Kelman and Spence, 2004, defines 3 non- physical flood actions: chemical actions, nuclear actions and biological actions. Nuclear actions are very unlikely to occur in the case of Valkenburg, but the other two actions might have an influence on the design considerations of the structure.

The most likely chemical load is that of the water itself. Water might cause corrosion in steel bridges and in the rebar of concrete bridges. In the case of concrete, the durability class of a bridge is XD3 (chloride attack, wet/dry cycles), which has the highest concrete cover (30-55 mm depending on structural class) according to Eurocode 2. This is due to bridges being subjected to de-icing salt in the winter. For steel bridges, the main concern is corrosion. The steel can corrode when coming into contact with water and salts. To prevent this, steel can be coated by galvanizing or painting the steel.

Summation physical loads

With basic numerical values of all forces known, the ULS and SLS design load for the bridge can be determined. Firstly, the obtained values are summed up in table 4.1 below.

Table 4.1: Overview of loads on bridge

		Characteristic distributed load kN/m^2
Vertical loads	Snow	0.7
	Wind	± 0.441
	Traffic	34
	Hydrostatic	-5.10
	Hydrodynamic	-7.13
	Self-weight	9.04
Horizontal loads	Wind	0.637
	Hydrostatic	21.44
	Hydrodynamic	3.24
	Impact	-

To test the capabilities of a bridge, the most unfavorable scenarios have to be checked. In this case, two scenarios are of importance:

- Load combination 1:
  - * Self weight
    - Prefab slab
    - In-situ layer
    - Asphalt layer
  - * Prestress load
  - * Traffic load
  - * Wind load
- Load combination 2:
  - * Self weight
    - Prefab slab
    - In-situ layer
    - Asphalt layer
  - * Prestress load
  - * Hydrostatic load
  - * Hydrodynamic load

As stated before, checks can be done in SLS and ULS. The SLS check only takes characteristic values in account, while the ULS works with safety factors.

<u>LC1</u>
The vertical distributed SLS loading for LC1 is as follows:

$$G_k + Q_{1;k} + \sum(\psi_{0;i} \cdot Q_{i;k}) = 9.04 + 34 + 0.441 = 43.48 \text{ kN/m}^2$$

The vertical distributed ULS loading for LC1 is as follows:

$$\gamma_G \cdot G_k + \gamma_{Q;1} \cdot Q_{1;k} + \sum(\gamma_{Q;i} \cdot \psi_{0;i} \cdot Q_{i;k}) = 1.2 \cdot 9.04 + 1.5 \cdot (9 + 0.441) = 62.5 \text{ kN/m}^2$$

<u>LC2</u>
The vertical distributed SLS loading for LC2 is as follows:

$$G_k + Q_{1;k} + \sum(\psi_{0;i} \cdot Q_{i;k}) = 9.04 - 5.10 - 7.13 = -3.19 \text{ kN/m}^2$$

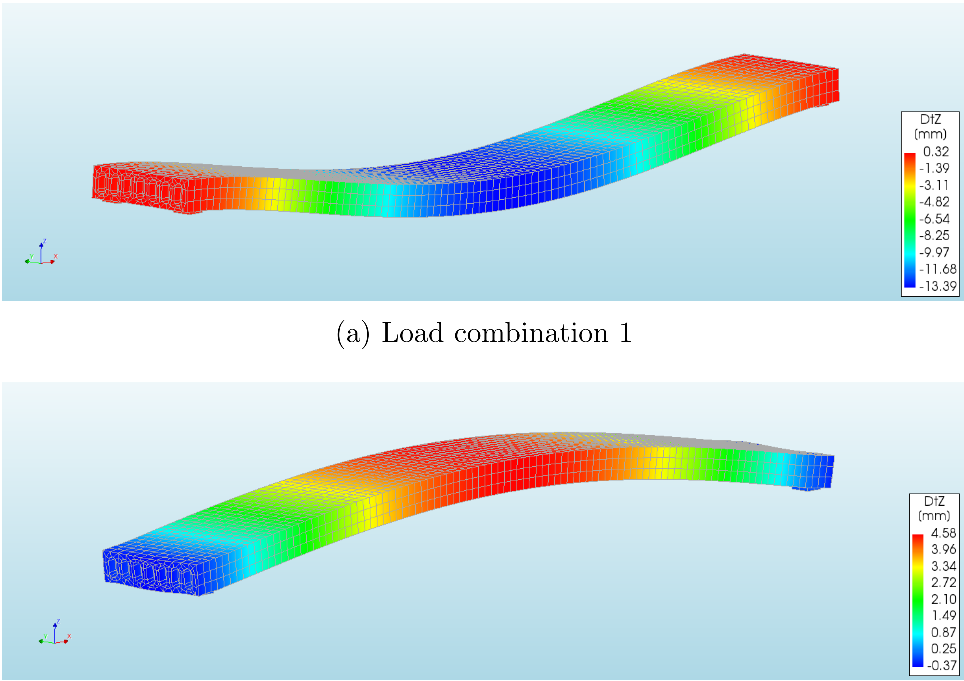
The vertical distributed ULS loading for LC2 is as follows:

$$\gamma_G \cdot G_k + \gamma_{Q;1} \cdot Q_{1;k} + \sum(\gamma_{Q;i} \cdot \psi_{0;i} \cdot Q_{i;k}) = 1.2 \cdot 9.04 + 1.5 \cdot (-5.1 - 7.13) = -7.50 \text{ kN/m}^2$$

Besides these vertical loads on LC1 and LC2, a prestress load, horizontal water load and impact load are present.

Finite Element Method

A good way to test a structural member is with the use of a Finite Element Analysis (FEA). With the dimensions, characteristics and loads of a member known, a model is set up and a linear analysis can be done to get the response of the system. The loads will be characterised in different Load Combinations, as stated above, to test the member in different scenarios. The model will be a simplification of reality and will be as follows. One hollow core slab is modelled, with dimensions of 1.2 x 0.32 x 10 meter. The concrete grade is C45/55. In the lower part of the cross section, 8 prestressed bars are modelled with a prestressing stress of 250 N/mm^2. The slab is modelled as semi-clamped to account for the bridge countering uplift.



(a) Load combination 1

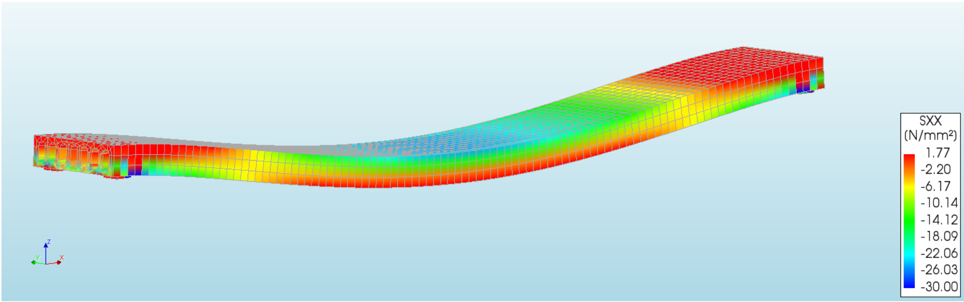
(b) Load combination 2

Figure 4.13: Deflection of heaviest loaded beam in SLS for two load combinations

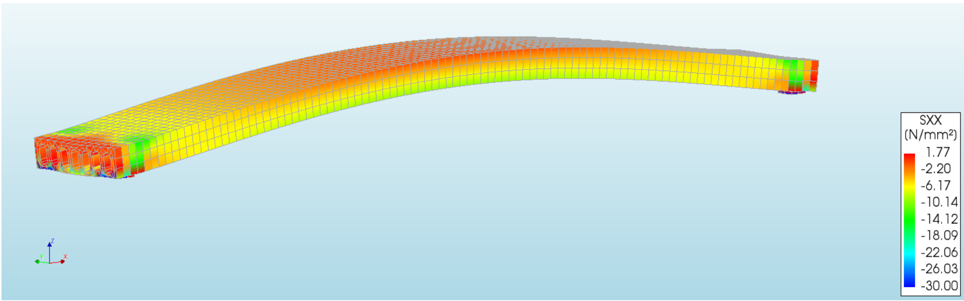
As can be seen in figure 4.13, the maximum deflections under LC1 and LC2 are 13.4 mm and 4.6 mm respectively. These values both don't exceed the threshold of a maximum deflection of

L/250. This comes down to a max deflection of 0.04 m = 40 mm.

The strength of both LCs will be verified on a basic level. The most important note is that a new bridge has to be capable of handling both 'normal' loads acting downwards, as abnormal loads acting upwards in case of a flood. The stresses in the slab have to be checked because tension is unwanted. On the other hand, the maximum compressive stress also should not be exceeded. For the chosen material, the design compressive resistance is 30 N/mm^2 and the tensile resistance 1.77 N/mm^2.



(a) Load combination 1



(b) Load combination 2

Figure 4.14: Stress distribution of heaviest loaded beam in ULS for two load combinations

As can be seen in figure 4.14 above, the compressive strength of the slab is not reached in both LC's. However, the tensile strength is reached in both LC1 and LC2 in mid span and at the ends in LC1. This means cracking would occur possibly leading to failure of the beam. This problem can be tackled with the addition of normal longitudinal reinforcement in addition to the prestressed reinforcement. This reinforcement would have to be added in the top of the cross section at the sides, and both in the top and lower part of the cross section at mid span. Furthermore, in the case of uplift the bridge needs to be constrained vertically. This will cause tension in the abutments. To account for this, tension piles could be added.

Costs

The costs of such an application are also of importance. A basic estimation will be made based on references and assumptions. The costs of a basic prefab bridge are estimated at 250$ per square foot (Wsdot, n.d.). With the bridge being 10 x 12 meters (1 square foot = 0.09290304 m^2) this would come down to €280,000. This would be the costs of a simple prefab bridge, excluding man hours. This is however not the entire picture, so costs have to be added. Firstly, this bridge design would have extra reinforcement, and abutments meeting the requirements for this case. These abutments will need to restrain the bridge in case of uplift, possibly needing tension piles. The costs of these extra reinforcements are negligible compared to the rest so will not be accounted for. The abutments accompanied with tension piles will have to be accounted for however. A rule of thumb for the costs of this type of abutment is 40% of the total manufacturing costs of the bridge (ArcerolMittal, n.d.). This would amount for around 115,000 €. Labour costs are said to be around 20% of total costs (Bridgit, n.d.). This would be around 80,000 €. Thus, the total costs of constructing a flat bridge like this would be 475,000 €. This is excluding the costs associated with designing the bridge.

Liftable bridge design

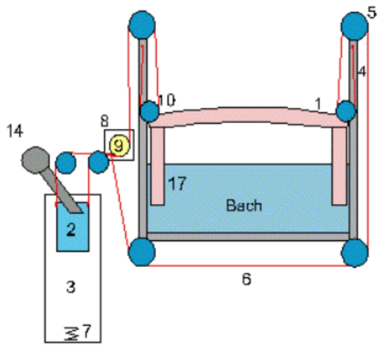
The second design option is that of a movable bridge. There are several options to move a bridge in case of high water, but in this section we will discuss that of a lift bridge.

In Brig, Switzerland, a similar problem arrived as the one described in this paper. In 1993, a river flowing through the city flooding leaving tons of damage and even the loss of two lives. The river had a problem with sediment transport in non-steep slopes, causing sedimentation. On top of that, trees and debris got stuck under a bridge basically creating a dam. The solution was reshaping the canal erasing bottlenecks of less steep parts. While designing this new canal, the redesign of the before mentioned bridge became important. The idea of creating a vertically liftable bridge arose (Saltina-Hubbrück, n.d.). This idea of a new liftable bridge had to fulfil the following requirement:

- Taking advantage of the problem
- No usage of external energy
- Simple mechanics for operational safety and service life
- Minimize maintenance and servicing

The idea that came to life was a vertical lift bridge which lifts with the help of counterweights filled with water. This means the mechanism only works in case of a flood, when the water reaches a certain level. The mechanism can be seen in figure 4.15b. The end product can be seen in figure 4.15a. The four corners of the bridge are attached to the lift mechanism by cables.

(a) Vertical lift bridge Brig



(b) Vertical lift bridge mechanics

Figure 4.15: Vertical lift bridge in Brig, Switzerland (Saltina-Hubbrück, n.d.)

The effect of implementing a vertical lift bridge in Valkenburg is mentioned in part 4.1.1. It is stated that without the addition of quay walls to fill the gaps, the discharge will be 122.5 m^3/s. In comparison, the maximum discharges of the current arch bridge system and conceptual flat bridge system are 65.3 m^3/s and 91.5 m^3/s respectively. With this information, the concept of a movable bridge seems promising.

With the effect of the implementation of a movable bridge known, the costs need to be considered. As a reference, the liftable bridge in Brig, Switzerland is used. The costs of this structure came down to 2.2 million Swiss franc, paid in 1997. These costs need to be converted to Euro and the present value needs to be calculated. The exchange rate at the time was 1 Euro = 1.64 Swiss franc. An average inflation rate of 3% is assumed for the last 25 years, giving the following calculation:

$$Costs = \frac{2{,}200{,}000}{1.64} * 1.03^{25} = € \, 2{,}800{,}000$$

It has to be noted that this is just the cost of construction without the design costs. Also, the costs of demolition of existing bridges is not taken into account, because these costs would be insignificant compared to the construction costs.

4.1.2 Raising quay walls

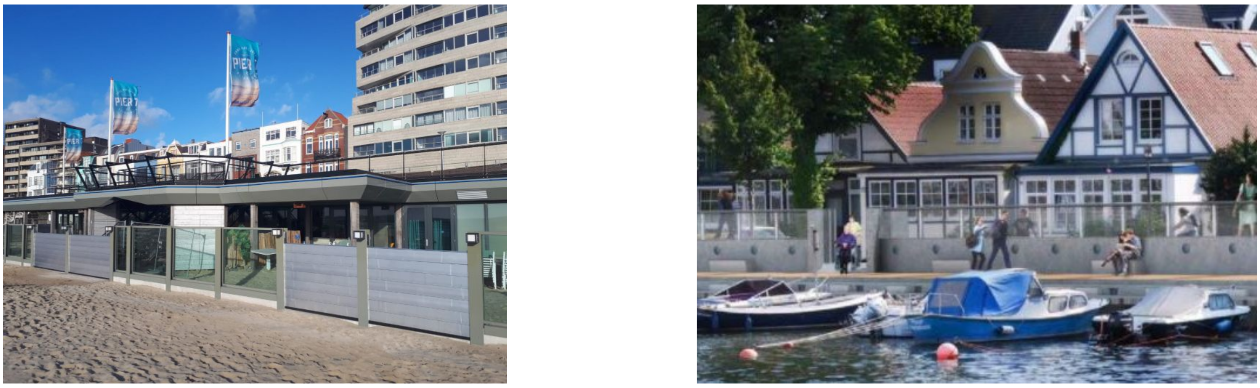
Raising the quay walls gives a higher protection against the water by increasing the maximum discharge through the Geul without overflowing. Higher quay walls are especially effective in combination with higher bridges. An objection for raising the quay walls is that higher walls limit the sight of the Geul, which is an important feature of Valkenburg. When inquiring the locals about this, most of them did not have a problem with raising the quay walls, they favoured safety over the aesthetics. Nevertheless, the ideal solution would be to limit the visibility of the higher quay walls. The solution for increasing the height of the quay walls must ideally meet the following requirements:

- The solution must be watertight
- The solution must be strong enough to resist the forces of the water
- The solution must be easy and quick to deploy
- The solution must be cheap and low in maintenance
- The solution should not be too visible

The cheapest and easiest solution is increasing the height of the quay wall using masonry. Ideally, the same type of masonry should be used as the existing wall to make it look authentic. The downsides are that the higher walls will completely block the visibility of the Geul, the walls might need to be reinforced when the new masonry will be placed on top of the existing walls.

Another option is removable or sliding barriers. The advantage is that these barriers are not or hardly noticeable, but they are generally quite expensive and high in maintenance due to the movable parts. A removable barrier has the problem that they need to manually installed at the threat of a flood, which might take too long as the inner city of Valkenburg has more than 2 km of quay walls that then needs to be heightened.

A promising option is that of glass panels for the quay walls. Glass panels hardly limit the aesthetics of Valkenburg, while providing good protection and affordability. The only problem of glass panels is that they need to be strong enough to withstand the loads on the panel, but glass panels have already been implemented in similar situations, as can be seen in figure 4.16.



(a) A glass storm flood barrier on the beach in Vlissingen (KWS, n.d.)

(b) An impression of a glass storm flood protection barrier in Warnemünde (Heyder, Frank en Paulu, Franziska, 2014)

Figure 4.16: Applications of glass barriers

An optimal solution would be a combination of permanent and removable barriers. The government needs to make a decision on the type of permanent barrier, glass or stone, based on costs and aesthetics. This permanent barrier needs to be complemented by removable barriers at places where permanent barriers are not an option, for example at the entrances to bridges, stairs or balconies.

In section 3.3 it is stated that there are gaps in the quay walls at the bridge openings, staircases and balconies that effectively lower the height of the quay walls. A simple solution to reach a higher safety level is to close these gaps at the threat of a flood. According to Waterschap Limburg, 2017, water already inundates into the center of Valkenburg at a water height with a 1:25 return period. The local water board was planning to install or provide some kind of water defence system at these aforementioned openings and implement them in their emergency flood plan (Waterschap Limburg, 2017). Some options for these temporary flood defences are explored below.

One option to fill in the gaps is to employ sandbags. Sandbags are cheap and easy to use, but take quite some time and effort to install. Moreover, the effectiveness of sandbags rapidly decreases with increasing water height (Reeve and Badr, 2003). Valkenburg is already quite dependant on sandbags and they have been used extensively during the flood of July 2021. The local broadcaster reported that the local government even momentarily ran out of sandbags just hours before the water height was at its highest (TVValkenburg, n.d.). This highlights another problem with sandbags: the effectiveness of sandbags depends on the timely distribution of the local authorities of the sandbags (Reeve and Badr, 2003), which might become a problem when demand is already high.

Another option is to use temporary flood barriers, for example planks connected to a wall mount, as can be seen in figure 4.17.

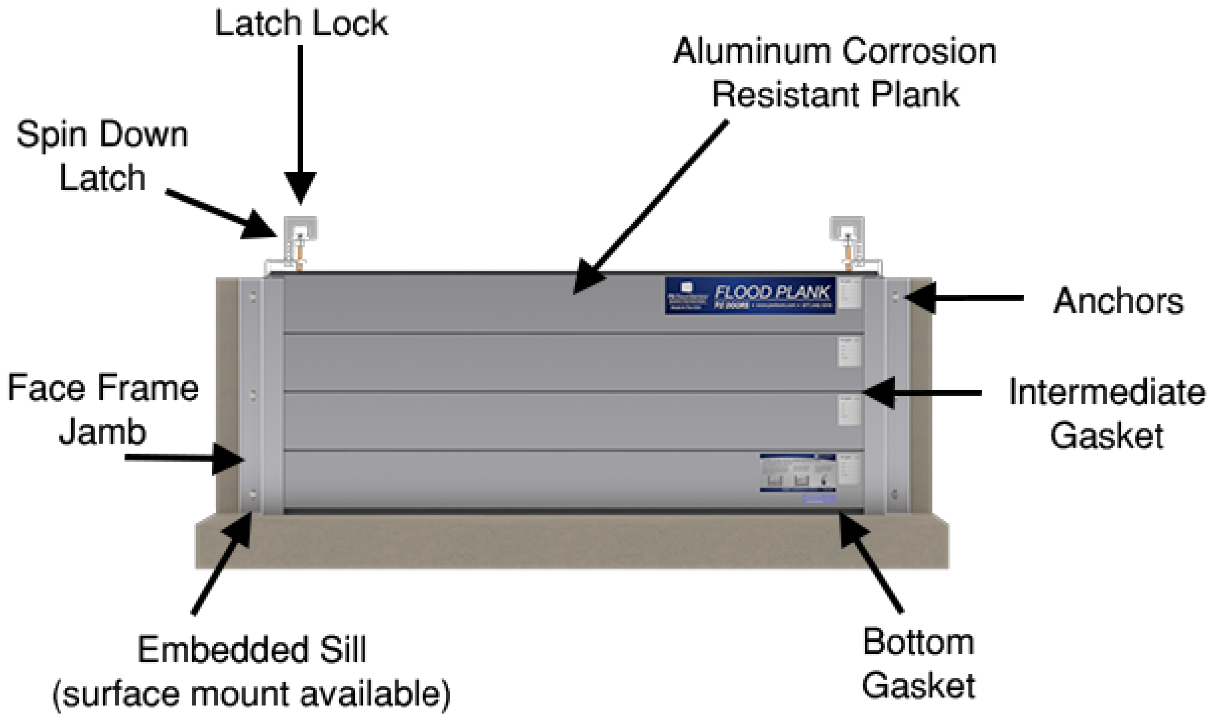


Figure 4.17: An example of a removable flood barrier (HydroDefense, n.d.)

These only require wall mounts and removable intermediate support columns are only needed for larger spans. These removable supports only need floor connectors that can be closed with a hatch when not used, so that traffic can easily pass over. The wall mounts that can even be placed at the river side of the quay wall so that they are hardly visible. The only problem with these walls is that they require a smooth surface to seal to. This can be achieved by simply casting a concrete strip into the existing pavement. If this is unwanted or too expensive, a layer of sandbags or a rubber strip might also be used to limit the leakage. This system or something similar can also be used for larger spans, for example at the square in Valkenburg in figure 4.18a which only requires more removable supports.

There are also locations in the city center where the facade of residencies is incorporated into the quay walls, as can be seen in figure 4.18b. Similar removable, sliding or lifting flood barriers can also be installed in front of windows or doors of residencies and or businesses. This is especially important when the quay walls will be raised, as even more vulnerabilities in the quay wall become apparent when doing so.

(a) The Bogaardlaan in Valkenburg. Only one side of the river is protected by a quay wall

(b) The facade is sometimes incorporated in the quay wall

Figure 4.18: Openings in the quay wall

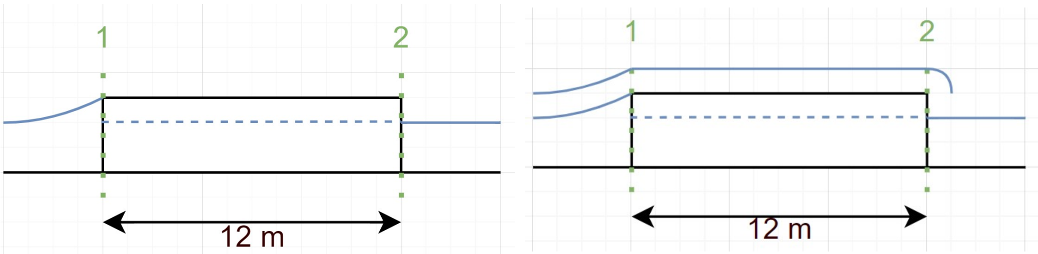
An important aspect of choosing the type of quay wall are the costs. Masonry walls similar to that found in the city center of Valkenburg (likely sandstone or limestone) costs somewhere between €100-€200 per square meter, with most sources estimating the cost at around €150. But these costs are estimates for walls meant for gardens or sheds. The quay wall is a lot thicker, and therefore we multiply these costs by 2. Of course, finding the original type of masonry to match with the existing wall can vary hugely in price, depending on the current availability. The removable barriers like the one in figure 4.17 vary enormously in price. The price mainly depends on the maximum height, span, if it can be bolted to the floor or to walls, and the ease of deployment. But a first order estimate of the costs would be between €750 - €1500 per meter at a height of 1.5m (RPDB Southern Tier Central, 2020). The length of the quay walls in total is roughly 2500 meters. The total length of the gaps is a little more difficult to estimate, but when mainly looking at the bigger gaps, for example at bridges and at the Bogaardlaan (see figure 4.18a), the total length is about 250 meters. But when adding the smaller parts, for example at the entrances to peoples homes or gardens, the estimate will likely be closer to 300-350 metres.

The labour costs should also be taken into account. The estimates for these are 40% of the material costs. The total costs for filling the gaps in the quay wall will therefore be between

€315,000 - €735,000. The total costs for raising the quay walls by 25cm *and* filling the gaps would be between €490,000 - €1,085,000. It should be noted that this estimate is not very accurate and that the actual costs might vary. From these calculations, it can be found that the costs for the removable barriers is a very significant portion of the total costs.

Effect of raising quay walls on discharge

The situation is schematised in two flows in order to obtain the maximum discharge capacity when applying movable barriers. The first flow flows under the bridge as can be seen in 4.19a and the second one flows over the bridge as can be seen in 4.19b. In order to calculate this discharging capacity of the first flow, the Bernoulli Energy equation can be used like is done for the culvert in the example of Elger et al., 2014b. This can be done as the difference in head is limited like in the example and in the current situation. In this equation the energy head is taken at two cross-sections. Since energy loss will occur between the two sections, the energy loss has to be accounted for as well. The cross-sections taken are just before the arch bridge and just after it, and still have the dimension as described in section 4.1.1 (oval-shaped: 2.3m x 3.5m).



(a) Flow below the bridge (b) Flow over the bridge

Figure 4.19: Flow when the gaps between the quay walls are filled

The water level behind the bridge is estimated to be equal to the level of the bottom of the bridge. This is assumed based on videos of the recent floods of last summer. A snapshot of a video can be seen in figure **??**. Therefore, the energy equation for the first flow will be as following:

$$z_1 + h_1 + \frac{p_1}{g} + \frac{v_1^2}{2g} = z_2 + h_2 + \frac{p_2}{g} + \frac{v_2^2}{2g} + \Sigma H_l \tag{4.14}$$

The pressure will be equal on both sides ($p_1 = p_2$) as it is atmospheric at both cross sections. Also, the bed level is approximately equal on both sides ($z_1 \approx z_2$) as the distance is short and the bed slope is very mild. Therefore, the equation above reduces to:

$$h_1 + \frac{v_1^2}{2g} = h_2 + \frac{v_2^2}{2g} + \Sigma H_l \tag{4.15}$$

The energy loss in this equation can be split into the entrance loss, the exit loss and the loss within the bridge. while the in- and outlet losses are described with the minor loss coefficient. This results into the equation for energy loss:

$$\Sigma H_l = \frac{v^2}{2g}(\Sigma K) + h_{f,pipe} \tag{4.16}$$

The discharge through a pipe can be obtained with the Manning-Strickler equation due to its relatively low pressures:

$$Q = \frac{1}{n} A R_h^{\frac{2}{3}} S_0^{\frac{1}{2}} \tag{4.17}$$

With:

$$R_h = \frac{A}{P} \text{ and } S_0 = \frac{h_f}{L}$$

When implementing the formulae above and rewriting this for the head loss in the pipe, this gives:

$$h_{f,pipe} = \left(\frac{Q}{\frac{1}{n}A\frac{A}{P}^{\frac{2}{3}}}\right)^2 * L \tag{4.18}$$

When applying the continuity equation ($v = Q/A$), all equations can be filled into equation 4.15 to obtain the maximum discharge. The minor loss coefficients are estimated based on literature of loss in transitions of pipe flow (Elger et al., 2014c). These coefficients are dependent on the diameter of the pipe and the angle. By using the book, K_{in}-value is assumed to be 0.28 and K_{out}-value is assumed to be 0.12. The Manning's Roughness Coefficient n is estimated based on literature like Memon et al., 2015, and Elger et al., 2014a, and is again assumed to be 0.014.

Now, the discharge is left as the only unknown when filling in the values stated above, and can therefore be obtained. This results in a discharge of 44.0 m^3/s for the flow beneath the bridge in the northern part in Valkenburg. This is the lower than the capacity just before the culvert was full, as is expected from figure 4.2 and the pressure is limited. The calculation is obtained via the python-code which can be found in figure **??**.

Now, the flow over the bridge still has to be added. This flow is schematised as Strickler-Manning and therefore, can easily be obtained like done in equation 4.1. The top layer of the bridge is relatively smooth. However, objects like the handrail increase the roughness. Therefore, the Manning coefficient is again assumed to be 0.014. The other variables remain unchanged resulting into an additional discharge of 20.3 m^3/s. The total discharge can now easily be obtained by adding the flow below and over the bridge resulting in a discharge of 64.3 m^3/s in the northern branch. This results into a total discharge capacity of 91.9 m^3/s in Valkenburg corresponding to a return period of approximately 213 years. It is important to note that the turbulence caused by the bridge can result into higher local water levels and therefore, a more careful study must be conducted in order to see whether a local increase of the quay wall might be necessary.

Additionally, also the quay walls can be raised further. The discharge capacity over the bridge is increased to 28.4 m^3/s. The total discharge capacity through Valkenburg when raising the quay walls will approximately be equal to 103.2 m^3/s corresponding to a return period of 869 years. These numbers are solely aimed as a first-order approximation given the many uncertainties and estimations in dimensions, coefficients and other numbers. When comparing it to last summer's floods, the return periods were of the same order and the water level observed was slightly higher. Though it gives a good indication of the effectiveness of the possible measures.

The same can be done for the situation with flat bridges. The water level at the exit is assumed to be the same as for the exit. It might not be a perfect representation of reality but is acceptable for a first-order approximation. The K_{out}-value is assumed to be 0.10 and disappears from the equation while the K_{in}-value is assumed to be 0.20. The energy balance can now be filled in and a discharge of 55.4 m^3/s is obtained which gives a total discharge of 79.1 m^3/s. When the gaps are closed, the same extra discharging capacity in the northern branch is added as for the arch bridges: 20.3 m^3/s. This results into a total discharge of 108.1 m^3/s corresponding to a return period of almost 1709 years. The return period is even equal to more than 10129 years when applying flat bridges, closing the gaps between the quay walls and also raising the quay walls by 25 centimeters.

4.1.3 Water tunnel

By the end of August 2021, a concept to prevent flooding was suggested which has not been implemented in The Netherlands yet. This concept focuses on increasing the discharge through an alternative route below ground level: a *water tunnel*, as can be seen in figure ??. According to Jeroen de Leeuw, this concept can perfectly be used in a valley area like Valkenburg (Witteveen & Bos, 2021). The philosophy of the concept is that excessive discharge flows in the inlet of the tunnel upstream from Valkenburg, and comes out of the tunnel downstream from the city. By doing so, the water that would normally cause a higher water level, is now discharged underneath Valkenburg, which will result in a reduction of water level in the city centre of Valkenburg. In other words: the peak in water level will theoretically speaking be restricted. De Leeuw states that a tunnel with a diameter of 3.5 meters can carry approximately 30 cubic meters water per second from one end to the other. If a tunnel with these dimensions would be constructed, and the excessive discharge would be greater than 30 cubic meters per second, the water level in the city centre would start rising again. However, the water level that would occur with the water tunnel constructed, would be smaller than the water level without water tunnel.

The concept is based on the gravity flow of water as can be seen in figure 4.20. As long as the inlet has a higher elevation than the outlet, water can only flow into one direction. Additional inlets as in the figure are not required because only little additional water enters the Geul between the entrance and exit point of the tunnel.

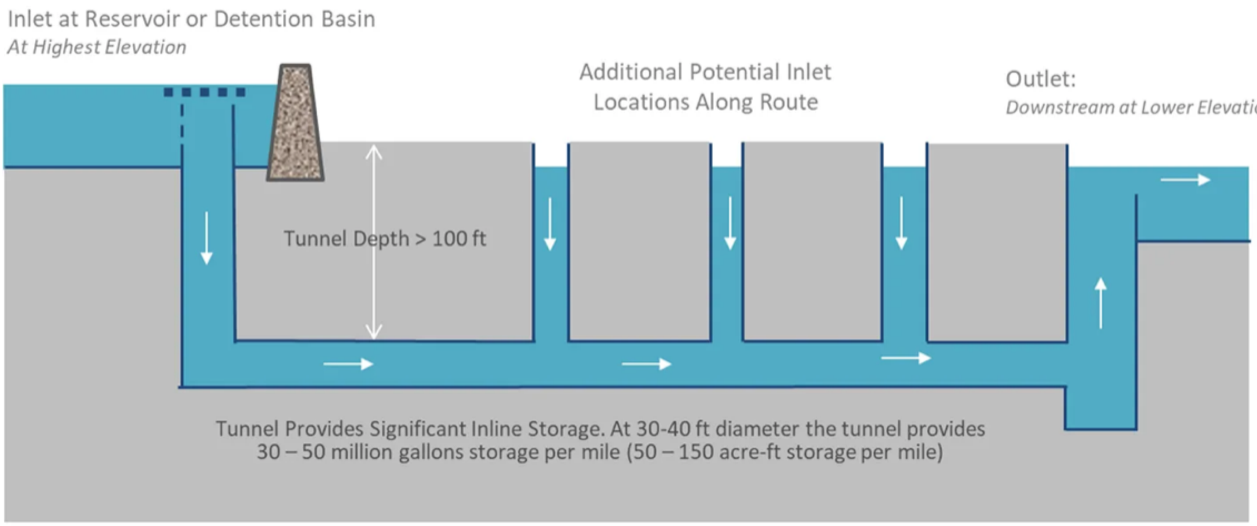


Figure 4.20: The concept of a water tunnel (Rehak, 2019)

Water tunnels have already been applied in several cities in the United States of America, for example in Dallas, San Antonio, Austin, Houston, Chicago and Washington D.C.. Switzerland and Japan are other countries that already use flood tunnels and flood reservoirs. Some of these tunnels are built to reduce sewer pollution into rivers, while others are solely built to flatten the peak water level during high water events. After hurricane Harvey caused floods in Houston in 2017, Harris county decided to protect the city with a water tunnel. First of all, they had a look at comparable flood tunnels (Freese and Nichols, Inc., 2019). Two of them, are meant for storm water:

- Mill Creek/Peaks Branch/State-Thomas Drainage Relief Tunnel – Dallas, Texas
    - * Length: 8.1 kilometers

  * Diameter: 9.1 meters to 10.7 meters
  * Discharge capacity: 424 m^3/s
  * Costs: 265 USD/ft/ft (2770 €/m/m)

- San Antonio River Tunnel – San Antonio, Texas
  * Length: 4.9 kilometers
  * Diameter: 7.3 meters
  * Discharge capacity: 189 m^3/s
  * Costs: 625 USD/ft/ft (6540 €/m/m)

These two examples perfectly show what well engineered tunnels are capable of. The costs of these two tunnels, and 48 other tunnels, have been plotted in figure 4.21. We assume that the water tunnel in Valkenburg will be constructed using the *one-pass lining system*, since most tunnels in this research have been constructed in this manner.

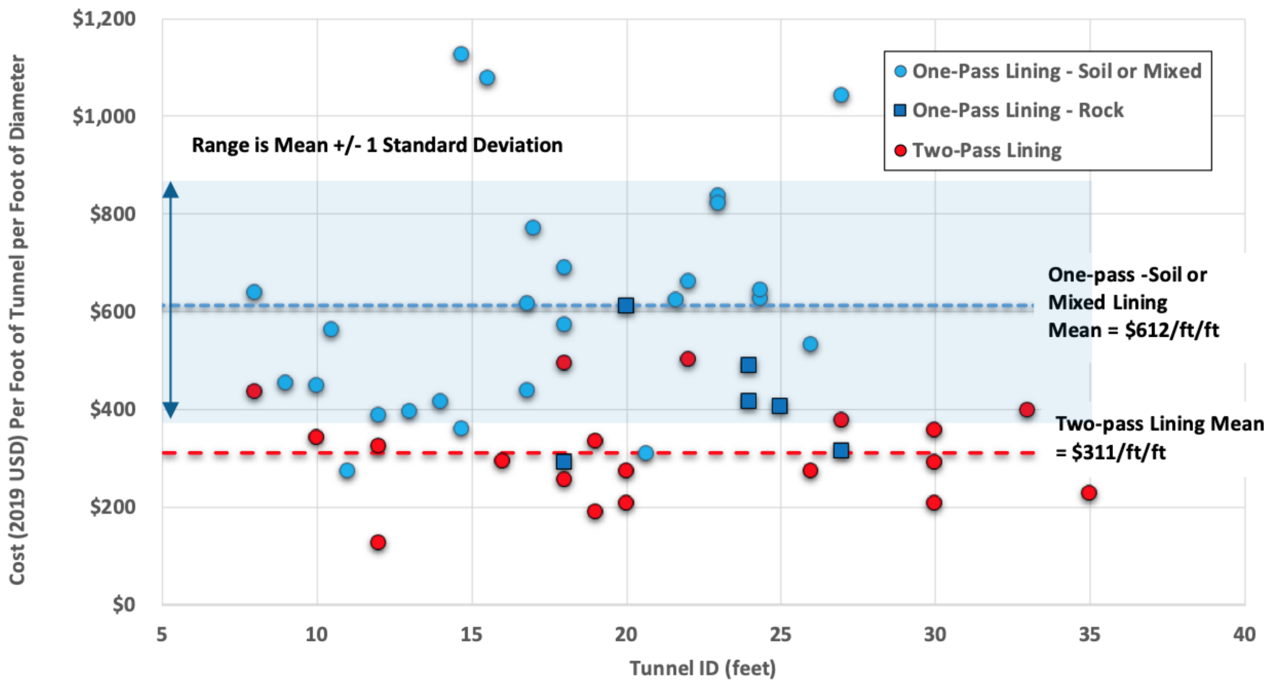


Figure 4.21: Costs of 50 tunnel project in the USA and Canada, on the x-axis: diameter of tunnel [feet], on the y-axis: costs generalised to length and diameter [USD 2019] (Freese and Nichols, Inc., 2019).

The mean price for one-pass lining tunnel systems [€ 2022], as can be calculated from figure 4.21, is 6407 €/m/m. This value is obtained by normalizing the total costs by length and internal diameter. This normalized value assumes that a doubling in diameter would result in a doubling in costs. We question this relationship, since a twofold increase in diameter would result in an area that is four times the original area and thus, four times more ground that has to be excavated. However, we continue our estimations with this normalized value. When calculating costs for potential concepts, the *AACE Estimated Accuracy Range* class 5 is used as in Freese and Nichols, Inc., 2019, since it accounts for a concept screening design. This class holds a lower range cost estimation of -30%, and an upper range cost estimation of +50%.

It is assumed that all excessive discharge greater than 71.2 m^3/s flows into the tunnel, as long as the excessive discharge is smaller than the discharge capacity of the tunnel. When water flows into the tunnel, two possible scenarios are possible. The first scenario holds that the tunnel is not completely filled up with water. Water levels in the city centre of Valkenburg will therefore not rise to a level where flooding will occur. This scenario could be computed using open channel flow formulas. The second scenario entails that the water tunnel is filled up with water. All space in the tunnel is used to discharge water downstream, and discharge capacity cannot be further enlarged. A pressurized situation is created which is the bottleneck discharge capacity. When discharges in the Geul would keep rising, the city centre would start to flood.

A python code has been written in order to find the pressurized discharge capacity of different conceptual tunnels. This code uses the Bernoulli equation to find the speed, and thus the discharge capacity of the tunnel. The loss coefficients that are assumed can be found in table **??**. The python code itself, can be found in figure **??**. Six conceptual tunnels have been designed, and their corresponding dimensions and parameters are shown in table 4.2 and 4.3. The length of concept 1 to 3 is 800 meters, since this is the length Witteveen & Bos proposed. The length of concept 4 to 6 is 1300 meters, because there is more space to build a decent outlet structure further downstream. The discharges given in these tables correspond to a Darcy friction coefficient of 0.017. This coefficient is based on the Moody chart with a relative roughness of approximately 0.0002 and a Reynolds number in the order of 10^6. Discharges for other Darcy friction coefficients, can be found in figure **??**. The tables show that an increase in diameter significantly increases the discharge capacity. The tables also highlight that and an increase in length does not change the discharge significantly due to a slight increase in differential head. However, this increased length increases the costs of the tunnel concept with a factor 1.625.

Link to safety level

Using the sum of the pressurised discharge capacity of a tunnel concept and the discharge where no damages occur, a safety level can be linked to each of the concepts. The return period of the summed discharge can be found using an adapted code based on the generalised extreme value analysis code from section 3.3.1. These return periods are also shown in table 4.2 and 4.3.

Table 4.2: Water tunnel concepts 1 to 3, length = 800 meters, differential head = 4 meters

Parameter	**Concept 1**	**Concept 2**	**Concept 3**
Diameter [m]	2.5	3.5	4.0
Discharge capacity [m^3/s]	13.9	29.6	39.8
AACE Class 5 *Range LL - HH [€]*	8,969,800 19,221,000	12,557,720 26,909,400	14,351,680 30,753,600
Safety level [Y]	100	635	2580

Table 4.3: Water tunnel concepts 4 to 6, length = 1300 meters, differential head = 5 meters

Parameter	**Concept 4**	**Concept 5**	**Concept 6**
Diameter [m]	2.5	3.5	4.0
Discharge capacity [m^3/s]	13.4	29.1	39.5
AACE Class 5 *Range LL - HH [€]*	14,575,925 31,234,125	20,406,295 43,727,775	23,321,480 49,974,600
Safety level [Y]	95	595	2469

4.1.4 Executing 4-step measure like Meerssen

Some options to regulate the flow in the Geul can be thought of. The neighbouring village of Meerssen came up with a 4-step plan to tackle the problem of high flows. These steps involve 4 different areas with measures per area as follows (Water in balans, 2020):

1. **Country side**
   Due to the high area of farmland in the region, a relatively high run-off flows towards the river, further increasing water levels. The goal of Meerssen is realising 10 mm of extra infiltration in order to lower the run-off. This can be achieved by applying the following principles (Water in Balans, 2020a):

   - Changing the land use in forest or grass as can be seen in figure **??**;
   - Creating zones of grass or bushes on the sides of the fields;
   - Using sandbars in between crops in parallel with the contours of elevation. This will also result into compaction due to the wheels of the material in parallel with the contours of the elevation;
   - Improving the quality of the soil. This can for example be done by leaving the green waste on the ground. This will prevent compression of the soil and has a positive influence on the ecosystem in the soil which will enhance infiltration in the ground. Also, soil fertilisers can be used;
   - Creating buffers where water can better infiltrate in the ground;
   - Increasing the roughness of the bottom surface by using planting e.g. grass;
   - Planning activities careful to prevent soil compression during wet conditions;
   - Roughening of the subsurface which will break the droplets;

2. **Urban area**
   The run-off in the rural area can not only be decreased, but the same goes for the run-off in the urban area. This can be done by:

   - Enhancing infiltration. Preventing rainwater from entering the river Geul can help to reduce its discharges.
   - Creating local storage. The municipality of Meerssen wants to achieve decoupling of the river Geul of 50% in Ulestraten and 50% in Meerssen.
   - Delaying run-off.

3. **House owners**
   In order to reduce the damage, the self-reliance of home-owners can be increased. When citizens are given enough time to prepare, the following things can be done (Water in Balans, 2020b):

   - Making houses waterproof;
   - Shuts preventing the water from entering;
   - Proper urban planning for new-built buildings.

4. **Water system**
   The water system can be altered to reduce the damage by:

   - Increasing the discharge capacity reducing bottlenecks;

- Increasing large-scale water buffering. Meerssen increased the buffering capacity by 90.000 m^3.

The approach mentioned above is applied to small-scale streams. However, de Geul contains a relative big catchment area compared to the small streams which lead to the flooding in Meerssen. This means that not only the difference in discharge is bigger but also the high flood peak takes longer. This means that all measures should be applied on a bigger scale. The most drastic measure is to alter the water system to reduce bottlenecks. However, due to the buildings right beside the canals, this is not possible in Valkenburg.

The measures described in the section house owners do not reduce the probability of a flood taking place. The measures focus on mitigating the effects of the floods by for instance preventing the use of wooden floors and using shuts. These measures should be combined with a properly-functioning early-warning system in order for this to be successful. Else, citizens do not have the time to install shuts for example. Also, it is essential to create awareness for the possibility of flooding. The conducted survey indicated a lack of awareness and therefore, it is essential to create social acceptance. This would require involvement of the locals. As stated in section 3.1, the locals want to limit the damage and therefore are likely to cooperate. Strategies to create more social acceptance are described in greater detailed in section 4.2.1.

The other measures mentioned focus on delaying, retaining and storing the water upstream or locally. The big advantage of this approach is that it does not only increase the safety level of Valkenburg, but it also positively affects other downstream cities. This approach would require cooperation with the farmers for the rural area as the measures described should be executed on the property of the farmers. This can result into conflicts as their business activities are influenced by them. Therefore, cooperation with the farmers is required in which the urge of the measures are explained and negative consequences for the farmers are compensated. This is currently already done as can be found in the annual report of 2020 of Water in Balans.

A runoff reduction can also be achieved in the urban area. Measures can be taken in both in the public and private domain. Increasing the amount of runoff can be achieved by increasing the green area and implementing green roofs. Cooperation is required when these measures are performed in the private domain. Citizens should be aware of the possible measures and in order to stimulate them further subsidies can be implemented.

To see whether flooding with a 100-year return period can be prevented by reducing the runoff, a first-order estimation is performed. The amount of water that is required to be stored or infiltrated upstream can be calculated by estimating the duration of the high water and taking the difference in the discharge between the certain return periods. When assuming an approximately constant high water duration of 24 hours and assuming no damage for T = 25 years, it would lead to a required reduction of discharge:

$$(Q_{100} - Q_{25}) * 24\,hours = (85.1 - 71.2) * 3600 * 24 = 1.2 \text{ million m}^3.$$

Increasing the infiltration and retaining capacity leads to a reduction of the run-off. This can be done by the methods described above for the urban and rural environment. However, an enormous volume of run-off should be prevented in order to prevent the high water flood. Assuming the same duration for different discharge and a linear relationship between runoff and discharge, it would require a decrease of the runoff-coefficient of more than 16% for the complete catchment area. Furthermore, research states that higher rainfall intensities leads to a higher runoff-coefficient (Mu et al., 2015). This is a logical result of the saturated soil resulting into runoff of all excessive water. Therefore, the required runoff-coefficient reduction will be less efficient and more measures will be necessary. It is clear that reducing the runoff-coefficient cannot solve the problem alone, since it would require extreme measures in the entire catchment area and therefore, should be combined with other measures like storage. An indication for the costs are obtained from a case study on the water quality of river Dommel where the investment costs for storage of 200,000 m^3 is estimated at €79 million (Benedetti et al., 2013). When applying additional storage to prevent flooding in the return period up to 100 years, it would require an investment of approximately €474 million. This storage can both be applied in the urban and rural area and can also be applied on a large- and small-scale. Besides the high costs, the feasibility is also relatively low as the land availability is low.

4.2 Non-technical aspects

In the previous sections of this chapter we have devised technical solutions to increase the safety level of Valkenburg. This section will discuss the non-technical measures that can be taken to increase the acceptance of a certain safety level.

4.2.1 social acceptance

Section 4.1 discussed technical solutions to increase the safety level. Besides implementing technical solutions to reach a certain safety level, it is also possible to reach more acceptance of the current situation by adjusting the expectations of the stakeholders.
The locals understandably prefer the safety level to be as high as physically possible, as was found in the survey (see section 3.2.3). When trying to get a more reasonable and quantifiable answer i.e. 1:100, almost all people still said that they wanted an unreachable high safety level. It was also found in the survey that people had little awareness of the current safety level of Valkenburg and that they felt very safe before the floods.

This all gives an indication that the locals are insufficiently educated on the dangers of living in an area like Valkenburg. The consequence of this might be that people are not well prepared when a flood occurs. When looking at videos of the flood and visiting Valkenburg after the flood, there were indeed indications that not all people were well prepared. A large number of houses have wooden floors and shuts were not always installed in front of doors and windows.

The safety board of Limburg- Noord (VRLN) has published a survey amongst 2219 people to evaluate the safety board's response to the floods. This survey's outcome contains some numbers that indicate the unawareness and unpreparedness of most people (Flycatcher, 2021):

- 63% of people did not worry about flooding prior to the flood, after the flood this was 37%
- 75% did not worry about evacuating prior to the flood, after the flood this was 50%
- 70% did not expect to have to evacuate, 14% did
- 89% was unprepared for evacuation, 82% did not consider evacuation
- 68% of surveyed people's homes were (partly) evacuated
- 96% of people did not use the government shelters
- 85% of people did not experience problems going back home after evacuation

- 25% of people received an NL-alert (an automated phone message in case of emergency), the information got a 6.1 out of 10
- The VRLN got graded a 6.7 for their work

At the moment of writing, the evaluation of the safety board of Limburg-South has not been published yet, but is expected to be published halfway January 2022. These numbers can of course vary for the south of Limburg, but these numbers can act as an indication.

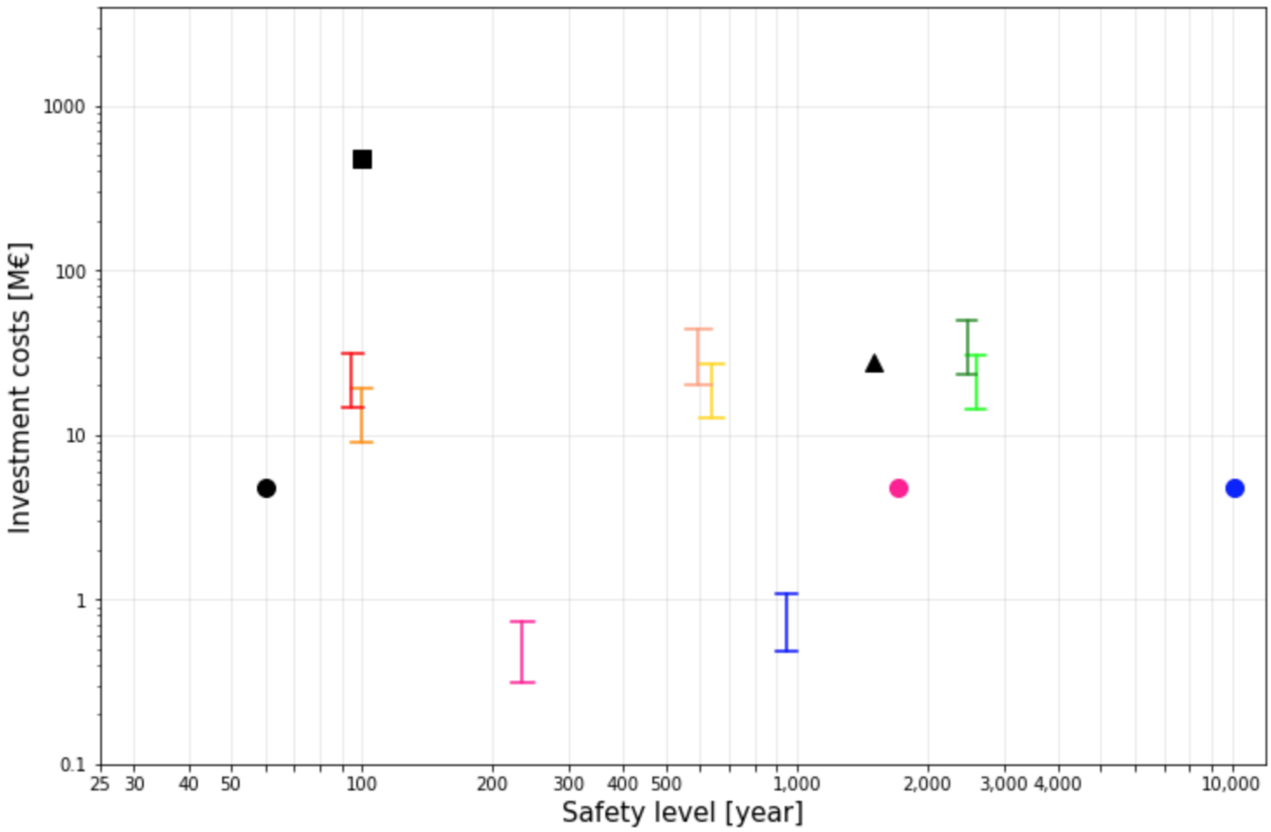
The lack of information is not true: there are numerous of public sources of information for the people in Limburg that are easily accessible. Sources like 'overstroomik.nl', 'crisis.nl', 'risicokaart.nl' or the website of the local safety board contain information on what to do in case of a flood and some give an indication of the potential threat of a flood in your location.

The abundance of readily available information and yet the lack of awareness of the locals on the subject seems to imply that the problem is getting the people to consume this information. Examples of ways to do this is to make an infomercial and broadcast it on TV and radio, or to make a booklet and send it by post to the people that live in an area that is prone to flooding.

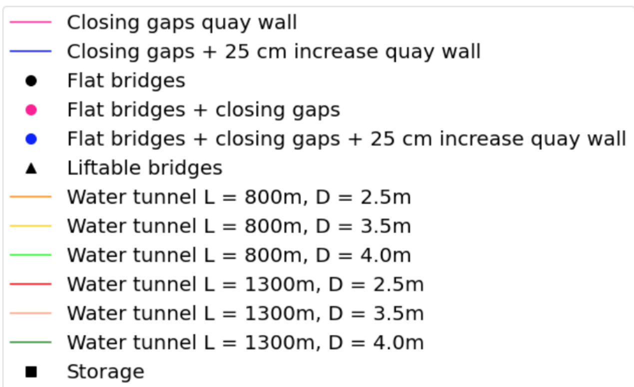
The benefit of getting this information to the people is that they are better prepared in case of a flood. Some damage and dangerous situations might be prevented when people evacuate on time and take preventive measures to keep the water out of their homes. Also, better awareness amongst the people might lower their expectations for the safety level to more reasonable numbers. When they are aware of the risks, they might better understand that not all floods can be prevented, and that this is unfortunately part of the risk of living in an area that is prone to flooding.

5. Results

In chapter 4, multiple solutions for increasing the safety level have been discussed, and their accompanying return period and costs have been estimated. The results are visualised in figure 5.1. Note that the axes of the graph are logarithmic.



(a) Investment costs and safety level



(b) Legend

Figure 5.1: The investment costs and corresponding safety level for all hydraulically and structural solutions

In figure 5.1 it can be seen that some solutions are more cost effective than others. For example, storage has approximately the same safety level as the water tunnel designs with 2.5m diameter, but the costs are far higher. In general, most solutions are somewhat on a line. The only big outliers are storage, which is not cost- efficient, and the solutions regarding the quay walls. Closing the gaps in the quay wall and potentially increasing the height of the quay wall are very cost- effective compared to the other measures. It is also interesting to observe the large effect of a small increase in the height of the quay wall. An even higher safety level can be achieved by increasing the height of the quay wall, at the expense of aesthetics and slightly higher costs. Solutions can even be combined to reach very high safety levels, but these are not included in the figure.

When a certain measure is implemented, Valkenburg is protected from floods with a discharge *up to* the accompanying safety level of that measure. But some measures can help reduce the damage that occurs when this discharge is exceeded, as indicated by the red line in figure 5.2. In this figure, the black line is kept the same as in figure 3.9. The amount of damage that a measure prevents is dependant on the type of measure chosen. For example, raising the quay walls prevents damage up to that safety level, but once exceeded, the water in Valkenburg will likely be at the same height as if the quay walls were not raised, and so the damage will be comparable. But for example the water tunnel will help prevent damages after exceedance of the safety level, because it will still decrease the amount of inundation, and therefore damage. While this type of prevention has not been taken into account in any of our calculations, it might be important as it can have a big influence on choosing the most cost- effective measure.

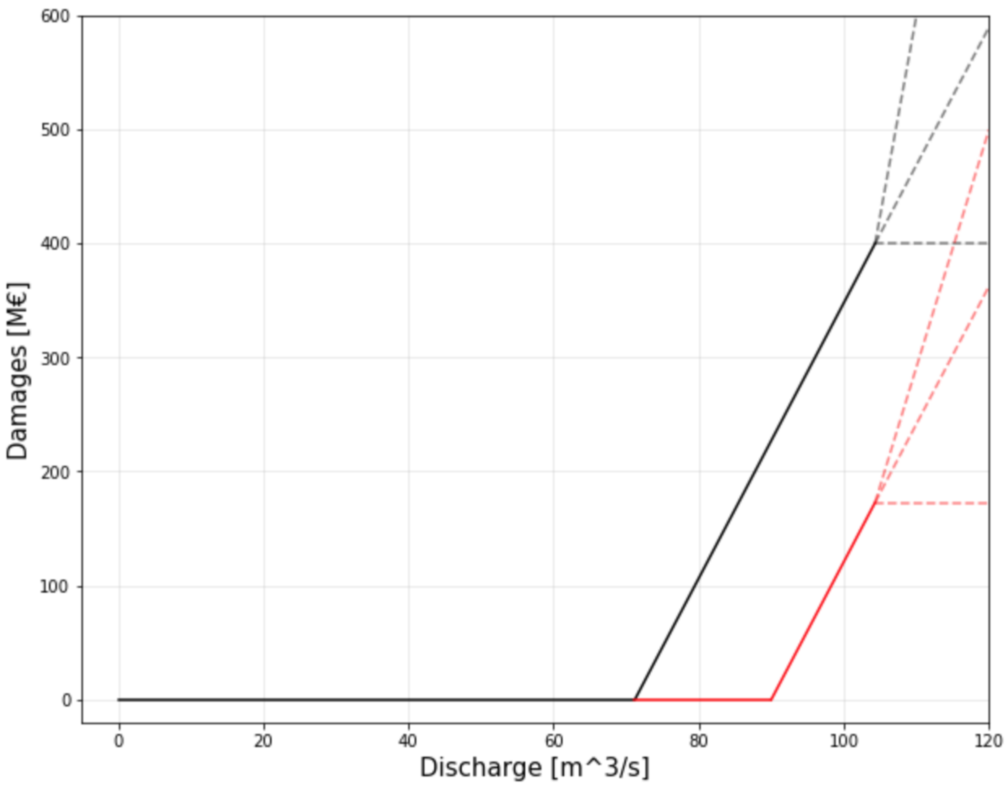


Figure 5.2: The implementation of a safety measure could lead to a reduction in damages

Another option to reduce damages is the one discussed in sections 4.1.4 and 4.2.1. Damage can effectively be reduced by increasing the preparedness of the inhabitants of Valkenburg. When looking at figure 5.2, simple and cheap measures like placing shuts in front of doors and opting for stone floors instead of wooden ones will decrease the slope of the damage- discharge line to an extent. At relatively low exceedance of the design discharge capacity, some damage will likely still occur, but much less. And more importantly, when people are more aware and better prepared for flooding, they might better protect themselves, resulting in fewer casualties.

6. Conclusion

The July 2021 flood gave reason to investigate a possible inadequate flood risk management system of Valkenburg. It turns out the safety level of Valkenburg has a lower standard in comparison to the rest of the country, namely 1 in 25 years compared to 1 in 100 years or higher. The basis of this safety level lies in simple back of the envelope calculations. The same reasoning is used for the determination of possible extra safety measures, which were written off following brief calculations in a Cost-benefit analysis. Solely due to the enormous damages of 400 million, these decisions seem questionable. In addition, individual risk norms are possibly not met. With an individual risk calculation using assumptions and data of the recent flood, casualties, and inhabitants, it is shown to likely be bigger than the required individual risk of $\frac{1}{100{,}000}$. Furthermore, a survey showed that inhabitants of Valkenburg were mostly not aware of the flood risk of their city. This survey also showed that their sense of safety related to flooding decreased after the flood. Most of the people questioned demanded a higher safety level than the current standard. They would even be open for an increase in tax to realise this improvement. To double the safety level of Valkenburg, fifty percent of people would triple their water board tax, compared to twenty percent for doubling the tax and even thirty percent for paying nothing extra. Increasing the safety level using elevated quay walls, and thus decreasing the aesthetics of the city, was the most represented opinion. The entrepreneurs who rely on tourist based income however, do not prefer this option due to the loss in aesthetic value.

A Generalised Extreme Value-method (GEV) is applied to obtain a return period for different discharges. This resulted in probability and cumulative density functions for the city of Hommerich. With this information, certain return periods could be coupled to discharges for Hommerich, which could then be multiplied by a factor to obtain the same for Valkenburg. With these discharges known, damages were coupled to them.

Several solutions have been worked out and checked on their effectiveness. The first solution is related to the redesign of bridges in the city centre. Firstly, the effect of different bridge designs on the discharge capacity is analysed. Three options are described, namely (already existing) arch bridges, flat bridges and a liftable bridge. The outcome of this rough analysis is that the discharges of the three bridge systems are 65.3, 80.1 and 107.2 m^3/s respectively. Secondly, a case study is done on the redesign of a collapsed bridge, using a flat bridge design. Requirements and loads are described and worked out to perform a basic structural analysis. From this analysis it can be concluded that a new bridge has to be designed to withstand both conventional as in-conventional loads. The latter takes into account the uplift and impact load caused by water in case of a flood, which means that more focused studies on the abutments are necessary. The suggested design consists of prestressed hollow core slabs topped with in-situ concrete and an asphalt layer. These would also need to be restrained in a way to prevent the bridge floating away. The abutments needed for the bridge should be installed with the addition of tension piles to counter the possible uplift. The hollow core slabs need normal longitudinal reinforcement in addition to the prestressed reinforcement, to withstand tension forces that could induce cracking in . This reinforcement would have to be added in the top of the cross section at the sides, and both in the top and lower part of the cross section at mid span. The cost of such a structure is estimated at €475,000. Furthermore, the idea of a liftable bridge is elaborated on. This bridge would lift in case of a flood, using the weight of the water. As said before, this type of bridge would have a great discharge capacity of 107.2 m^3/s, but is quite costly (2.8 million euros).

The second aspect is related to raising the quay walls in the city centre. Several alternatives are possible, including 'normal' masonry walls, glass walls and removable flood barriers. Both the costs as the effect on the discharge of raising the quay walls and/or filling the gaps are determined. Firstly, the costs for just filling the holes come down to €315,000 - €735,00, whereas the costs of both filling the holes and raising the quay walls 25 cm come down to €490,000 - €1,085,000. Secondly, the effect of filling the gaps is calculated to result in a discharge of 91.9 m^3/s, compared to a discharge of 103.2 m^3/s when filling the gaps *and* raising the quay walls 25 cm. This is coupled with the existing arch bridges. When looking at flat bridges, the discharge becomes 79.1 m^3/s when raising the quay walls, and 108.1 m^3/s when raising the quay walls *and* filling the gaps.

The third aspect is related to the implementation of six possible water tunnel concepts with different design parameters. The six different concepts differ in length and diameter leading to different discharges and corresponding safety levels. With this information, the municipality has to make a choice regarding the length and diameter. A bigger diameter gives a significantly higher safety level, but accompanied with higher costs.

The fourth aspect is related to implementing parts of Meerssen's 4-step approach. This comes down to making changes in the country side, urban area, water system and at the house owners' side. Here it is focuses on retaining, delaying, and storing of precipitation. It is shows that these measures itself will not significantly increase the discharge capacity. However, the accompanied costs of the storage come down to 474 million euros.

Non- technical solutions are also proposed that focus on making people more aware of the risk they are exposed to. This mostly means distributing information to people to which could eventually lead to more acceptance and thus more pleased citizens.

Results are presented in chapter 5 where an estimation of costs and safety levels is done, showing that raising quay-wall is a very cost-effective measures. Furthermore, it can concluded that solutions like 'storage', water tunnel concepts with D = 2.5 m and just flat bridges are not optimal. These either cost relatively much or have a low effect. The water tunnel concepts with D = 4.0 m show to be a good alternative, with costs ranging from 15 to 50 million euro and providing high safety levels.

The main research question was:

How can the flood risk management system of Valkenburg be redesigned in order to improve the overall safety level?

The outcome of this research shows that there are multiple measures that can be taken. When looking solely at the costs and effects, flat bridges coupled with filled and higher quay walls seem like the optimal measure. Also big diameter water tunnels can significantly increase Valkenburg's safety level for a reasonable cost. However, costs and effects on the safety level are not the only important things to take into account. Raising the awareness and acceptance of citizens is also highly recommended.

7. Discussion

This chapter contains several remarks, assumptions and limitations on the project.

Firstly, it has to be said that the problem tackled in this report will not have a positive effect on the neighbouring areas of Valkenburg. The problem will merely be shifted outside the scope of Valkenburg. In particular Meerssen will have to deal with the changes done to the water defence system of Valkenburg. Their situation may even be worsened due to the lack of retention of water upstream is some alternatives.

Analysis
The GEV-analysis performed is based on data of 50 years. This already indicates that there is not a lot of data available for the extreme return periods. This means that the actual discharges for the extreme discharges are highly uncertain. Additionally, a factor was used to account for the difference of in discharge between Valkenburg and Hommerich. Also, the discharge of the 2021 flood massively affect the analysis. Since no proper measurements could be done for this event, the estimation for this event gives another uncertainty. This estimation was based on the equilibrium depth obtained with the skin-friction coefficient: c_f-coefficient. In hindsight, the manning-coefficient would be a better choice as this coefficient is independent of depth while the c_f-value varies a little. Multiple assumptions were made in the hydraulic calculations of the bridges, which might result in an overestimation of the discharges. Finally, the influence of future climate change is not taken into account. The intensity of rainfall is increasing leading to higher discharges, which should be taken into account.

Bridges
For the bridge design a couple remarks are of importance. Firstly, the estimation on the effect on the discharges contains a lot of uncertainties. The dimensions of the normative bridge were estimated via Google Maps, while a measuring would lead to a more accurate result. Additionally the roughness coefficients were estimated based on literature while calibration on the specific location would portray reality better. When doing a small sensitivity analysis, the roughness coefficient turns out to be extremely sensitive. Changing the roughness coefficient from 0.020 to 0.014 for the case of liftable bridges would for example change the return period from 108 years to 16331 years. Therefore, the all values should be calibrated to the situation in Valkenburg. Also, the flow is assumed to be steady while in reality, extreme discharges are non-steady for which the hysteresis can take place. This means that for the same discharge, different water levels occur. Therefore, the solutions might give a different water level than expected. The flow can become critical as well due to the bridges or other boundary conditions resulting into a lower water level than expected. This is not included as a hydraulic jump would bring it back to the normal water level resulting in possible flooding too. Finally, the discharge division is estimated. Besides being inaccurate, the estimation of a division implies a stable division for different discharges while it might vary in reality. A more detailed should be performed in order to make a more accurate prediction of the discharge capacity through Valkenburg.

The type of cross section is based on a commonly used alternative. There are also other alternatives not being investigated. Furthermore the traffic loads taken into account are used in a loading scheme based on assumptions. The true diaphragm action of the prefab slabs and in-situ layer remains unclear.

Quay walls
For the several alternatives of raising the quay walls, a few things need to be kept in mind. Firstly, the structural feasibility has not been checked. Concepts are merely mentioned to provide ideas for future implementation, knowing that these are merely tangible solutions. When incorporating these solutions, it is important that more studies are conducted on the suitability of these solutions. Finally, a better approach in hindsight would have been to describe the flow as broad-crested weir.

Water tunnel
The dimensions of the six water tunnel concepts are chosen in such a way that the applicability and capability becomes visible. A change in the assumptions for loss coefficients can however change the discharge capacity, and thus the return period of a concept. Another point of attention is the assumption that all discharge greater than the T = 25 discharge, flows into the tunnel . This is the ideal scenario since no floods would occur in the city centre. However, this might hardly or difficult achievable in reality.

4-step approach
Applying the 4 steps used in Meerssen was not feasible at a bigger scale. In order to give an estimation, the water storage was applied. Like mentioned, the feasibility of applying so much storage is low and the costs are high. In reality it would be an option to combine runoff reduction with storage. In order to do so, models have to be applied and these were out of the scope of this project. The thesis of Angela Klein gives a more detailed insight in this topic.

Saocial acceptance
While it is assumed that increasing social acceptance is a cost- effective way to reduce damages, this has not been proven in this report. When choosing to opt for social acceptance measures, it needs to be taken into account that these measures only work up to a certain extent. With high exceedance of the discharge capacity, simple measures will simply not be enough, and therefore social acceptance measures should be used in conjunction with hydraulic solutions. On the other hand, social acceptance also has a positive impact on the acceptance of flooding and the number of casualties, which might be important enough on their own to justify implementing social acceptance measures.

Results
The aim of this report was to show possible results for different desired safety levels. In order to clearly show them, the range of investment costs was plotted against the safety level. However, the operational costs can also differ for the different measures. Besides, a measure can be chosen not solely based on costs. Other factors, like described in the results, can play a significant role too. Additionally, some measures may reduce the damage while not altering the safety level. These measures can therefore be desired as well and can be checked with a CBA. Finally, some of the measures, like storage of water, may have a positive influence on downstream municipalities as well and therefore, some measures might be desired over other measures while this can not be seen in the graph shown in the results.

8. Recommendations

This study shows that when adjusting the safety level, it is important to listen to all these stakeholders and carefully explain them why certain decisions are taken. Also, it is also important to include an individual risk assessment as part of the process.

The return periods in this report are based on discharge level data of only a short period of about 50 years, and not for Valkenburg itself. When choosing to adjust the flood defence system, we would advise to use more concise data, and to incorporate climate change into the discharge and water level calculations. Also, a more careful study on the repartition of discharge between the two branches in Valkenburg should be conducted in order to be able to more accurately predict the safety level.

In the calculations for the discharge of the different bridge design, it was assumed that the limiting bridge would stay the limiting bridge after alteration. This could require adjustment of other bridges too. Therefore, it is recommended that the discharge for every bridge should be calculated separately, as it might be possible that some bridges are not limiting the discharge capacity and will therefore not have to be redesigned. Also, the coefficients used in the calculations to obtain the discharge should be calibrated for the situation in Valkenburg, as this might strongly influence the discharge capacity and therefore the return period.

To analyse the effect of the flood on a bridge, it is recommended a model is made. This means a 3D dynamic software model and/or a real life model. In these models, the effect of dynamic and static water load can be examined, and eigenvalue of the bridge can be checked to check the susceptibility of flutter.

When raising the quay walls, it is advised to check whether the quay wall will be strong enough to endure the increased horizontal water pressure. Also, when it is decided that raising the quay walls using stone is not preferred due to aesthetics, we recommend that (parts of) the quay wall will be replaced by glass panels.

When opting for installing removable barriers, the municipality should incorporate placing the barriers into their emergency plans so that someone is directly responsible for this task. It is also important that the responsible persons are trained to install the barriers and that they know where the barriers are stored. Furthermore, we recommend the barriers to be built a bit higher at the location of the bridges, since here the river is most likely to inundate due to increased local turbulence and backwater.

When opting for the water tunnel, we would advise the municipality and other involved parties to account for the high uncertainty in costs that goes with tunnel boring. When designing the in- and outlet of the tunnel, we recommend that the engineering company and municipality collaborate to find an appropriate spot that do not interfere with future spatial plans.

When all bridges are replaced with liftable bridges, the middle part of the city centre becomes inaccessible. Therefore, the people who are present in this part of the centre, need to be evacuated prior to the lifting of the bridge. We would advise the municipality to revise the evacuation plans in order to prevent people from being stuck.

From the results of the survey we found that not all inhabitants of Valkenburg are well- informed about the potential risks of living in such an area. We therefore recommend that the local government or waterboard should provide more information on the matter.

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