Design of a flood bypass tunnel for Valkenburg aan de Geul

Operation and hydraulic design to reduce the risk of flooding

MSc Thesis

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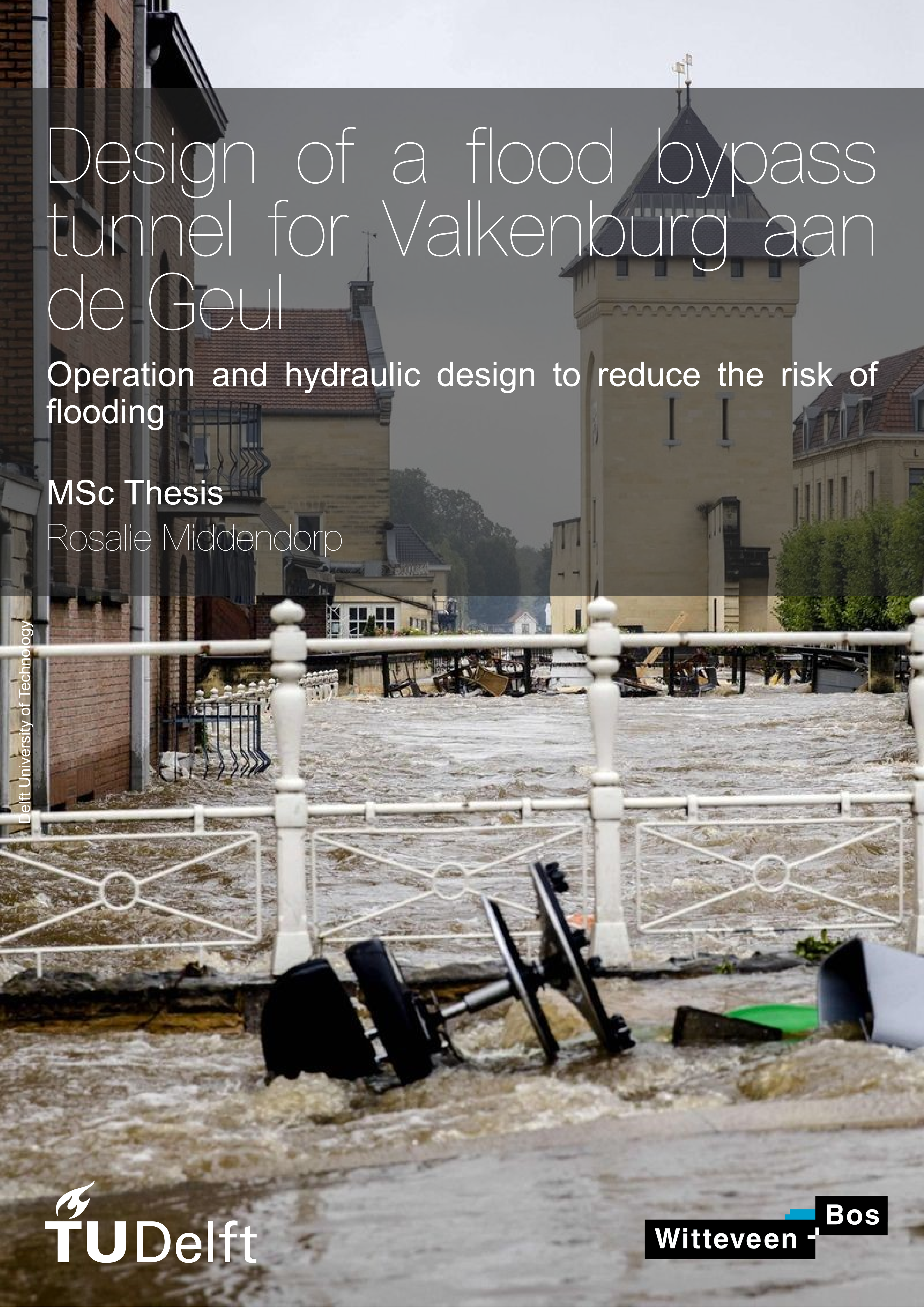

Delft University of Technology

TU Delft

Witteveen + Bos

Design of a flood bypass tunnel for Valkenburg aan de Geul

Operation and hydraulic design to reduce the risk of flooding

by

R.M. Middendorp

to obtain the degree of

Master of Science

in Hydraulic Engineering

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Cover: Flooding Valkenbug by Der Spiegel (2021)

Preface

This thesis is the final requirement for the degree of Master of Science in Hydraulic Engineering at the Faculty of Civil Engineering and Geosciences of Delft University of Technology. It focuses on the design of a flood bypass tunnel for Valkenburg aan de Geul. I am grateful for what I have learnt during this process and I would like to thank the following people for their contribution to this process:

First, thanks to Jeroen de Leeuw for the great idea of a flood bypass tunnel for Valkenburg and mainly for guiding me through the thesis process. Furthermore, I would like to thank Marcel Wauben for his expertise in the hydraulic aspect of the project and for keeping me focused on the objective of the research. Also, a thank you to the other colleagues of Witteveen + Bos who helped me along the way.

Secondly, my committee; Davide Wüthrich, Martine Rutten and Bas Jonkman, thank you for your expertise, critical remarks, and support throughout my thesis.

Additionally, I would like to thank the Delta future labs Limburg team for their brainstorming sessions, motivation and trips to Limburg. I would also like to thank Gijs van den Munckhof and Maurice Smeets of Waterschap Limburg and Ben van Eijsden, Jana Chvalovska and Daisy Blaauw from Gemeente Valkenburg aan de Geul, for their contribution to my thesis.

Lastly, I would like to thank my friends and family, especially Stephan and my mom, for keeping me motivated throughout my studies.

Rosalie Middendorp
Delft, February 2023

Summary

Between July 12 and 15, 2021, extreme rainfall in Luxembourg, Belgium, Germany, and the Netherlands caused severe flooding. The total damage in the Netherlands is estimated to be between € 350 and € 600 million, with Valkenburg aan de Geul suffering the most damage. The economic and tourist heart of this city is located in the lowest part of the valley, therefore developing a flood reduction measure is challenging due to the limited space and restrictions in this location to protect the culture and nature.

In previous studies, different solutions were proposed for reducing the flood risk such as flood walls in Valkenburg; however, due to the locational difficulties, many options are difficult to implement except for the option of a flood bypass tunnel. A flood bypass tunnel has never been applied in the Netherlands, however; it is applied in mountainous countries. A flood bypass tunnel rapidly conveys floodwaters through densely populated areas. For Valkenburg, a flood bypass tunnel will reduce the flood risk locally while taking up little space and keeping the historic district intact.

Recent research did not look further into the opportunity of a flood bypass tunnel as a flood measure assuming it would be too expensive (Asselman & van Heeringen, 2023). However, different studies concluded that a flood bypass tunnel is a viable option (Van Dijk, 2022; Kallen et al., 2022; Leijser & Nijhof, 2022). Nonetheless, a flood bypass tunnel has never been applied in the Netherlands, and there is no prior research detailing the hydraulic and operational applicability of a flood bypass tunnel for Valkenburg aan de Geul. The main objective of this thesis is therefore to develop a hydraulic flood bypass tunnel design for Valkenburg aan de Geul which should operate to reduce the flood risk.

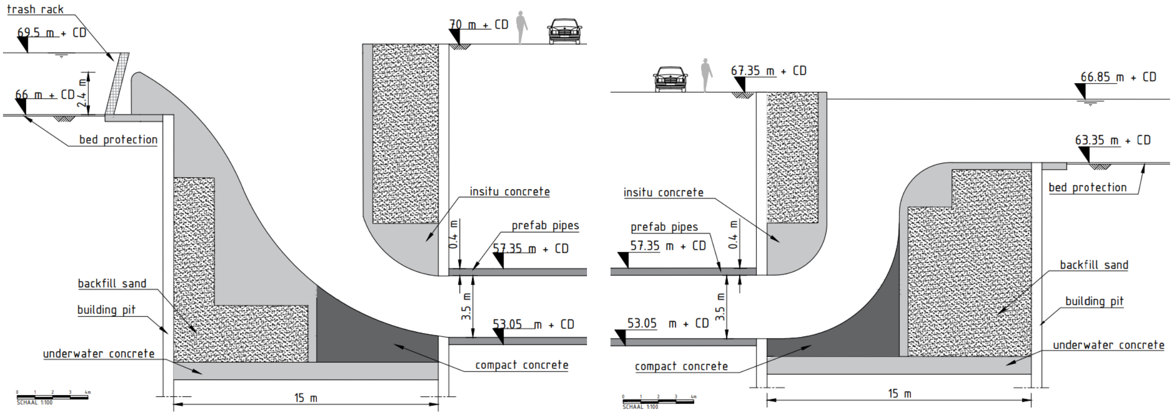
The objective is reached by combining the design approach for hydraulic structures with a Systems Engineering approach. First, the river system and its environment are analysed. Secondly, the basis of the design is determined. Thirdly, two alternative designs are developed from the two different types of reference projects: a passive and an active flood bypass tunnel.

The first design alternative is based on a passive flood bypass tunnel. It consists of a 2.4-meter-tall and 24.5-meter-wide Ogee weir, two tunnel tubes of 3.5 m diameter, and allows for a maximum discharge capacity of 55 m^3/s. Once the tunnel is filled, the flow is pressurised. After a flood, the remaining water will be pumped out. A co-current channel is designed to prevent water from refilling the tunnel from the outlet side in a non-flooding situation.

The second design alternative is based on an active flood bypass tunnel. It consists of four vertical moving flat gates at the inlets and outlets, two tubes of 3.5 m diameter, and allows for a maximum discharge capacity of 58 m^3/s. The water level will be controlled by the flat gates using a system for early automatic detection of flood hazards. The tunnel is always filled, thus pressurised. After a flood, the gates at the in- and outlets will close off the tunnel from its environment. The gates are tested twice a year, during which the system is flushed and refreshed.

The two design alternatives were evaluated according to six weighted criteria and ranked using a multi-criteria analysis in discussion with the municipality (gemeente Valkenburg aan de Geul) and the water-board (waterschap Limburg), and a cost analysis was conducted. The first design alternative with the passive flood bypass tunnel was selected due to its high reliability and serviceability, which were highly valued, and low maintenance costs, compared to the second design alternative. See Figures 1, 2, 3.

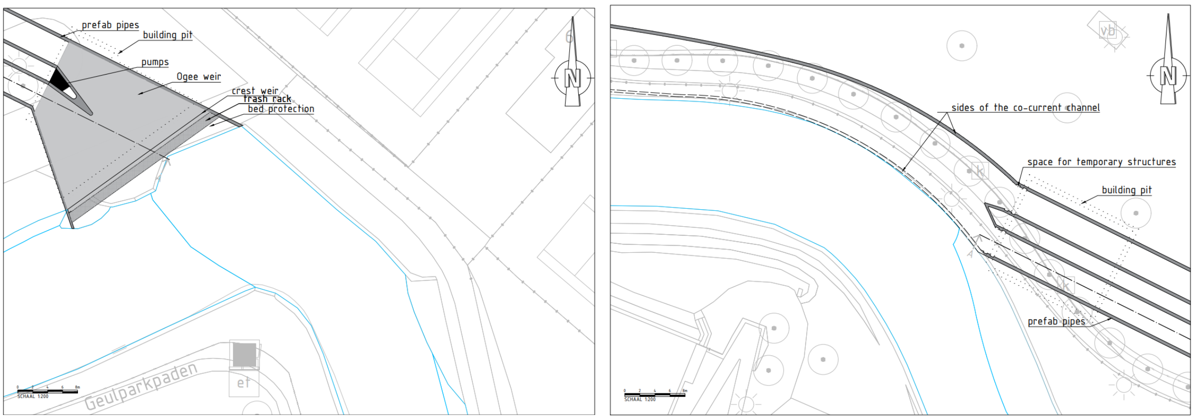
The flood bypass tunnel reduces the flood risk based on discharge reduction from an estimated once every 19 years to once every 250 years in the current climate. This accounts for uncertainty due to climate change and ensures flood risk reduction in the future. The flood bypass is only active when a flood is impending; hence the water remains to flow through the Geul and does not interfere with the cultural heritage and tourism of Valkenburg aan de Geul. Due to the cost-efficient pipe jacking method, the total construction cost is approximately €40 million with an estimated yearly maintenance cost of €100 k.



(a) Cross-section of the inlet

(b) Cross-section of the outlet

Figure 1: Cross-sections of the inlet and outlet of the selected design alternative



(a) Top view of the inlet

(b) Top view of the outlet

Figure 2: Top view of the inlet and outlet of the selected design alternative

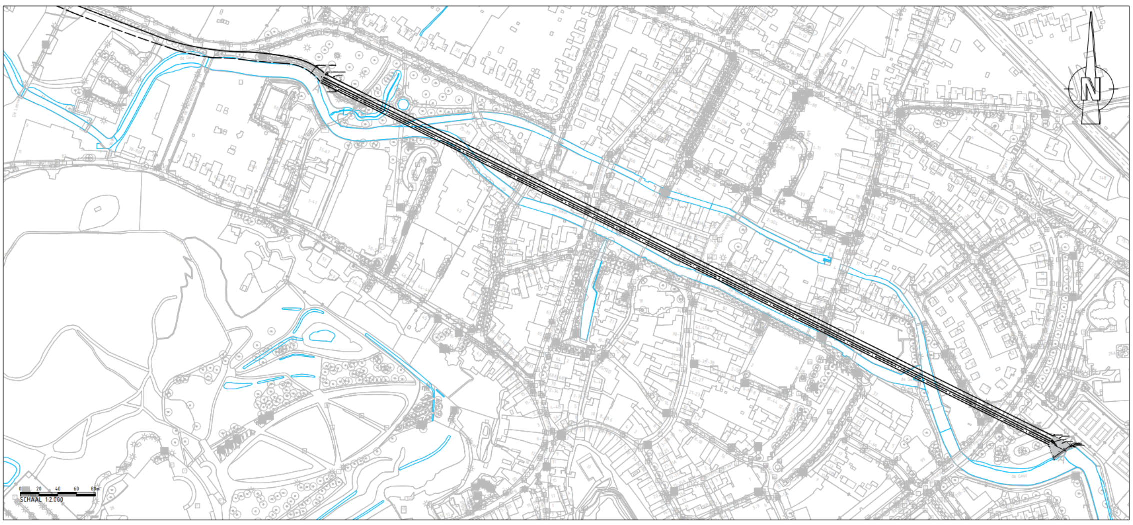


Figure 3: Top view of the selected design alternative

Contents

Nomenclature

Symbol	Definition	Unit
A	Area of the flood bypass tunnel	$[m^2]$
A_{ogee}	Constant	[-]
B	Constant	[-]
Δb	Difference in bed level elevation	[m]
b	Width of the weir	[m]
c	Weir coefficient for free flow	[-]
D	Diameter	[m]
F_D	Froude number	[-]
f	Friction parameter	[-]
g	Gravitation constant	[m/s^2]
ΔH_{bends}	Head loss through the bends	[m]
$\Delta H_{friction}$	Head loss caused by friction	[m]
ΔH_{inflow}	Head loss through the inflow	[m]
$\Delta H_{outflow}$	Head loss through the outflow	[m]
ΔH_{total}	Total head loss	[m]
$\Delta H_{trashrack}$	Head loss through the trash rack	[m]
H	The (upstream) energy level above the crest level	[m]
H_D	Design energy level above the crest	[m]
Δh	Difference in water level of location outlet until bedlevel	[m]
$i_{GeulatValkenburg}$	Gradient of bed slope of the Geul	[-]
K_t	Head loss coefficient trash rack	[-]
K	Constant	[-]
k	Roughness parameter	[m]
L	Length of the flood bypass tunnel	[m]
$L_{transition}$	Transition length from a square to a round tube	[m]
$l_{co-current}$	Length of the co-current channel	[m]
$l_{in-tooutlet}$	Length of river between the in- and outlet location	[m]
Q	Discharge	[m^3/s]
Q_{FBT}	Discharge capacity needed through the tunnel	[m^3/s]
Q_{flood}	Discharge of the flood with an estimate return period	[m^3/s]
$Q_{Valkenburg}$	Discharge capacity of the Geul in Valkenburg	[m^3/s]
Re	Reynolds number	[-]
S	Submergence	[m]
U	Velocity of flow without trash rack	[m/s]
V	Velocity	[m/s]
$W_{entrance}$	Minimal width of the inlet at the entrance	[m]
X	X-coordinate	[m]
Y	Y-coordinate	[m]
μ	Dynamic viscosity of water	[MPa $\cdot$ s]
ξ_{bends}	Bends loss coefficient	[-]
ξ_f	Friction loss coefficient	[-]
ξ_{inflow}	Inflow loss coefficient	[-]
$\xi_{outflow}$	Outflow loss coefficient	[-]
$\xi_{shapeinflow}$	inflow shape loss coefficient	[-]
$\xi_{shapeoutflow}$	outflow shape loss coefficient	[-]

Symbol	Definition	Unit
ξ_{total}	Total pressure loss coefficient	[-]
ρ_{water}	Density of water	[kg/m^3]

1
Introduction

This chapter describes the motivation of research, background, objective, scope, methodology and outline. The motivation of research is discussed in Section 1.1, followed by the background of the research in Section 1.2. The objective and scope are addressed in Section 1.3 and 1.4 respectively. Followed by the methodology in Section 1.5 and finally, the structure of the thesis is outlined in Section 1.6.

1.1. Motivation of research

Between the 12th and the 15th of July 2021, extreme rainfall in Luxembourg, Belgium, Germany and neighbouring countries, such as the Netherlands, lead to severe flooding, see Figure 1.1. The floods occurred in North Rhine-Westphalia and Rhineland-Palatinate in Germany and along the river Meuse and some of its tributaries in Belgium and the Netherlands (Kreienkamp et al., 2021). The flooding resulted in 196 fatalities in Germany and 42 in Belgium (NOS, 2021a). Besides the loss of life, considerable damage occurred to infrastructure, including houses, roads, communication, motorways, railway lines and bridges (Kreienkamp et al., 2021). The cumulative damage in Germany, Belgium and the Netherlands is estimated to be 38 billion euros, ranking the second most expensive natural disaster of 2021 (NOS, 2021a).

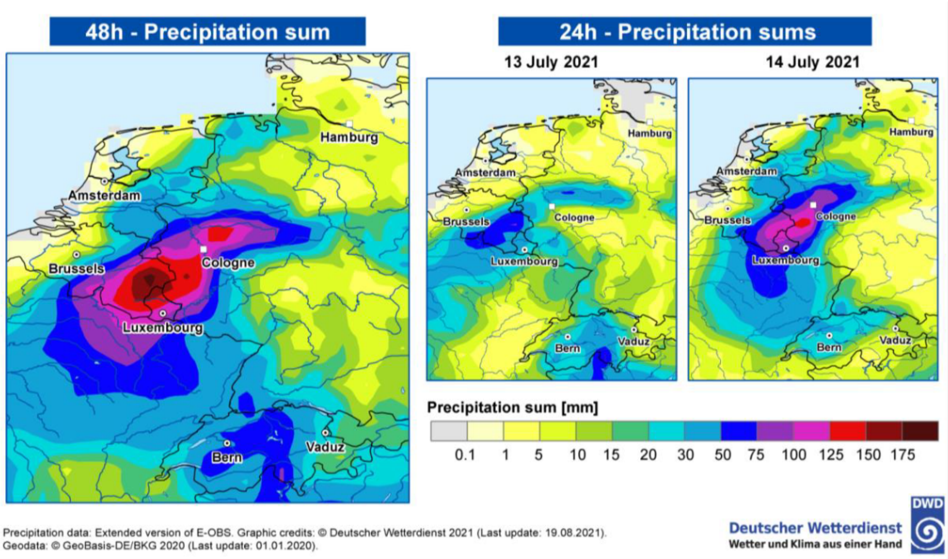


Figure 1.1: Precipitation accumulated over two days and accumulated over 24 hours (Kreienkamp et al., 2021)

The floods resulted in significant financial losses and damage in Limburg. An estimation of more than 2.500 houses, 5.000 inhabitants and 600 businesses were affected. The total damage in the Netherlands is estimated using the Dutch standard Flood Damage and Loss Model (SSM2017) and figures from international literature and comes down to be around € 350 – 600 million. The most significant damages are the damage to houses and businesses, business interruption, damage to infrastructure and crop losses. Next to economic damage, many people were evacuated and experienced health impacts, such as gastroenteritis, respiratory complaints, skin infections and psychological complaints (i.e. stress, concerns, and anxiety) (Task Force Fact Finding hoogwater 2021, 2021).

In the Netherlands, the most physical damage caused by the flood occurred at the river Geul. The city centre of Valkenburg was flooded and damaged the most (Task Force Fact Finding hoogwater 2021, 2021). The total damage of the flood in the municipality of Valkenburg aan de Geul is estimated to be 400 million euros (Binnenlands bestuur, 2021). Deltares also marked Valkenburg as one of the hotspots of the 2021 flooding in the Netherlands (Asselman et al., 2022).

Valkenburg aan de Geul is a picturesque medieval town located between hills in nature reserve 2000 Heuvelland. It is famous for marl and a busy tourism spot (Binnenlands bestuur, 2021). It is the second most popular tourist spot in South Limburg in terms of expenses after Maastricht. Around € 210 million is spent here each year by tourists (ZKA Leisure Consultants, 2018).

Due to the health impact on its civilians both physical and non-physical and the amount of damage and economic loss, together with the cultural heritage and tourism in Valkenburg aan de Geul, the focus of this thesis is on this region.

1.2. Research background

The flooding of the municipality of Valkenburg aan de Geul was caused by heavy rainfall; see Figure 1.1. The rainfall, in combination with other challenges and uncertainties related to Valkenburg aan de Geul makes developing a flood reduction measure difficult. The challenges and uncertainties are explained in Section 1.2.1 and Section 1.2.2 respectively, followed by possible solutions and the flood bypass tunnel in Section 1.2.3 and Section 1.2.4 respectively. This leads to the problem statement in Section 1.2.5.

1.2.1. Challenges

The challenges in this problem lay in the location of Valkenburg aan de Geul. The economic and touristic heart of the town, the city centre, is located in the lowest part of the valley (Visit Zuid Limburg, 2022). Here many protected cultural heritages are situated and the area around the town is a protected Natura 2000 area (Natura 2000, 2000). Therefore developing a flood reduction measure is challenging due to the limited space and the numerous restrictions in this location to protect the culture and nature. On top of that, the river originates in Belgium, thus the water crosses international borders. Therefore makes good transboundary water management is essential, which means in-cooperation of all parties as it regards international waters. This is difficult at the regional level at which the Geul is maintained as the establishment and creation of arrangements by the government bodies can be incompatible on both sides of the border. This can make cooperation troublesome (Keessen et al., 1996).

1.2.2. Uncertainties

Climate change causes faster recurrence of floods

Due to a warmer climate, the intensity of the rainfall for a 1-day or 2-day event increased, resulting in a faster recurrence of floods (Kreienkamp et al., 2021). Since 1980 the summer and spring are very dry, while the summer showers have increased steadily due to climate change. On top of this, the winter discharges are increasing (Tsiokanos, 2022).

Safety standard and insight in statistics of discharges unavailable

The safety standard in Valkenburg corresponds to a flood once every 25 years (Task Force Fact Finding hoogwater 2021, 2021). The summer flood exceeded the safety standard as the recurrence time of the summer flood was estimated to be once every 100 to 1000 years, but the precise recurrence time is uncertain as the statistics of discharges for the Geul are missing (Task Force Fact Finding hoogwater

2021, 2021). However, van Heeringen et al. (2022) made some estimates in their report which are used to estimate the return discharges in Chapter 2.

1.2.3. Possible measures

Numerous flood reduction measures can be considered to reduce the risk of flooding, however many of them are challenging to put into practice due to the location's aforementioned challenges and uncertainties. For example, an open channel bypass would be difficult to construct due to the low-lying city centre in the valley and a lack of space. Another possibility to reduce the risk of flooding is a retention area of 10.000.000 cubic metres, however, this will preferably also be located in Belgium (van Heeringen et al., 2022). As this crosses borders this would be more difficult to implement and also the lack of space is an issue. Another option would be to construct embankments along the river of 3 m high (van Heeringen et al., 2022). This would ruin the protected aesthetics of the picturesque town, which triggers the ongoing discussion of safety versus tourism says Har Frenken of Waterschap Limburg (NOS, 2021b). Therefore another solution was needed.

1.2.4. Flood tunnel bypass measure

De Leeuw & Mondeel (2021) proposed another solution, a flood bypass tunnel. This measure has never been applied in the Netherlands, however, it is applied in many different countries with mountains, e.g. Switzerland and USA. The bypass tunnel rapidly conveys floodwaters around the densely populated area and has a short length due to its cost. The tunnel does not retain water and needs the lowest amount of land since other structures can be located or built on top of it (Serra Llobet et al., 2021). The tunnel will reduce the flood risk locally while keeping the historic district intact, however, the flood bypass tunnel should be implemented together with other solutions for example a retention area to reduce the flood risk up- and downstream (De Leeuw & Mondeel, 2021).

The function of the flood bypass tunnel is to create a temporary bifurcation in the river, which increases the total discharge capacity of the system locally between the inlet and the outlet of the flood bypass tunnel. Due to the lower discharge in the river, the water level locally lowers at this part. A backwater curve will then develop following the locally lowered water level's free surface profile as transition curves (Mosselman, 2022).

The Geul river at Valkenburg has mild slopes and sub-critical flow, therefore, the downstream water level is normative. Upstream from the outlet, an M1-curve develops. This transition curve develops because of the deceleration of flow in the river before the outlet location due to the normative water level downstream, see Figure 1.2. Followed by an M2-curve upstream of the inlet until the initial water level is reached again. This transition curve develops because the flow accelerates before the inlet of the flood bypass tunnel due to the lowering of the water level, see Figure 1.2.

The actual backwater curves might differ depending on the discharge estimated to go through the flood bypass tunnel, see Section 7.1. The flood bypass tunnel is only temporarily in operation, therefore, it is assumed that there will not develop a new equilibrium of the bed level over time. Only initial bed level changes might occur, therefore, bed protection is needed.

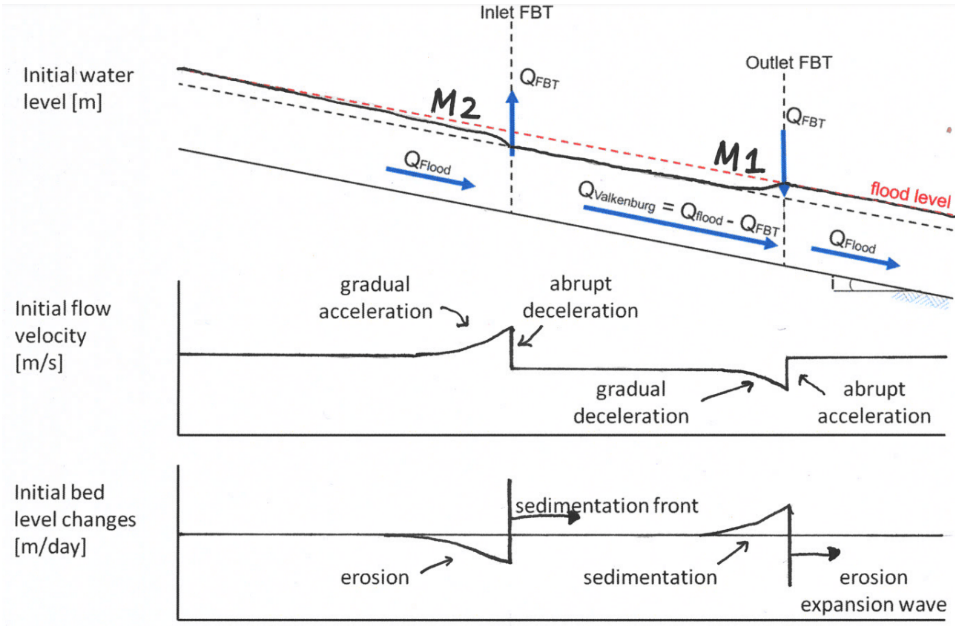


Figure 1.2: Backwatercurves ; based on Mosselman (2022)

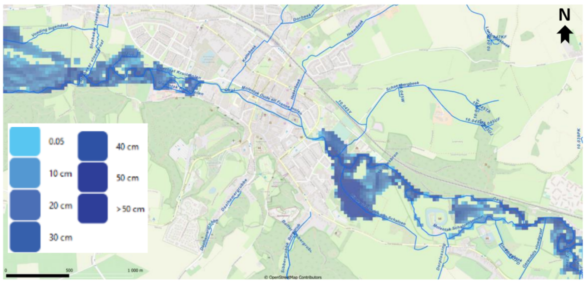
De Leeuw & Mondeel (2021) proposed constructing the flood bypass using the pipe-jacking method. This cost-efficient boring technique has a maximum inside pipe diameter of 3.5 m, therefore an estimated two pipes are needed. The flood bypass tunnel's inlet and outlet would be both constructed at public parking and the tunnel track would be around 800 meters, see Figure 1.3.

(a) Location of in- and outlet; edited of (ArcGIS Earth, 2022)

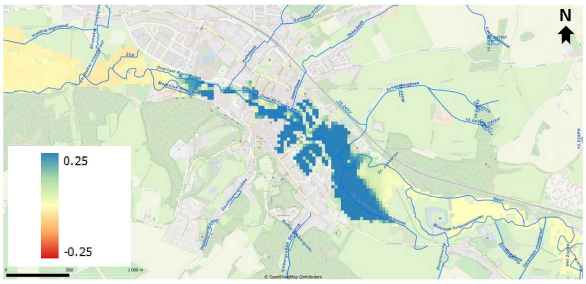
(b) Render of the tunnel design(De Leeuw & Mondeel, 2021)

Figure 1.3: Initial idea of the flood bypass tunnel

Other parties have already looked into the application of this possible solution of a flood bypass tunnel. van Heeringen et al. (2022) modelled the tunnel idea of De Leeuw & Mondeel (2021) with two pipes with a diameter of 3.5 m in SOBEK 1D model, and with the in- and outlet at the same location as De Leeuw & Mondeel (2021) envisioned. The discharge, depending on how easily the flood goes into the inlet, would result in 40 m^3/s as the tunnel would be pressurised. The peak discharge of the 2021 flood will be reduced between the in- and outlet by one-third. According to van Heeringen et al. (2022), this will not affect the upstream and will have minor negative effects downstream.



(a) Water depth of the flood after constructing two tunnel-tubes with a diameter of 3.5 m; edited of (van Heeringen et al., 2022)



(b) The difference in water depth between the current situation and after the construction of two tunnel tubes with a diameter of 3.5 m; edited of (van Heeringen et al., 2022)

Figure 1.4: Results van Heeringen et al. (2022)

Other TU Delft students researched different measures as well. Van Dijk (2022), Kallen et al. (2022), as well as Leijser & Nijhof (2022), concluded that Valkenburg's safety level could significantly increase for a reasonable cost due to a flood bypass tunnel with a big diameter. Van Dijk (2022) looked into a drilled tunnel with a diameter of 8 m, which was modelled in the SOBEK 1D model. The tunnel would cost between 112-207 million euros, due to the boring technique of the tunnel, but the effect of reducing the water level significantly (0.1 to 1.5 m) and the reduction of damages followed by the relatively low social impact. The discharge accounted for is 81 m^3/s through the tunnel to prevent flooding during the 2021 event.

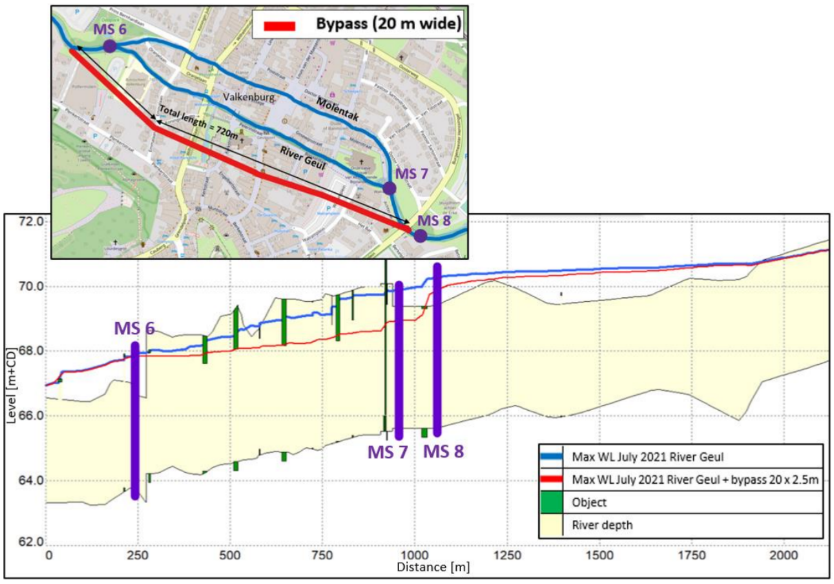


Figure 1.5: The result of a bypass which locally lowers the water level in Valkenburg (Van Dijk, 2022)

Kallen et al. (2022) looked into two different lengths of the tunnels 800 and 1300 m, with a corresponding head of 4 and 5 m, and different diameters of 2.5, 3.5 and 4.0 m. Depending on the variables, the discharge found varied between 13 and 40 m^3/s and a cost of 19 to 50 million euros. They concluded that Valkenburg's safety level could significantly increase for a reasonable cost due to a flood bypass tunnel with a big diameter. These two considered boring the tunnel instead of using the cheaper pipe-jacking method with a limited tunnel diameter.

Leijser & Nijhof (2022) looked at different lengths of tunnels and/or combined this with a traffic tunnel. Their preferred design was similar to De Leeuw & Mondeel (2021)'s design, thus without a traffic tunnel, however, there would be 3 tubes with a diameter of 3 m with a discharge of 78 m^3/s and a total cost of 34 million euro, using the pipe-jacking method. However, recent research did not look further into the opportunity of a flood bypass tunnel as a flood measure assuming it would be too expensive (Asselman & van Heeringen, 2023).

1.2.5. Problem statement

Van Dijk (2022); Kallen et al. (2022); Leijser & Nijhof (2022) concluded that a flood bypass tunnel is a viable option, however to date, there is no study that shows how the flood bypass tunnel should be hydraulically designed nor how the flood bypass tunnel should operate to reduce the flood risk in Valkenburg aan de Geul.

1.3. Objective

Following the research gap, the main objective of this project is to develop a hydraulic flood bypass tunnel design for Valkenburg aan de Geul, which should operate to reduce the risk of flooding.

The research question that follows from the objective is:

”What is the hydraulic design of a flood bypass tunnel to reduce the risk of flooding in Valkenburg aan de Geul?”

Keywords	Definition
Flood bypass tunnel	A flood bypass that is located underground in the shape of a tunnel
Hydraulic design	Design focused on the hydraulic aspects of the flood bypass tunnel
Reduce the risk of flooding in Valkenburg aan de Geul	A flood bypass tunnel will reduce the water level and thus the risk of flooding locally, so only in Valkenburg aan de Geul

This research question can be answered by answering the following sub-questions. The sub-questions correspond to the phases of the design approach, see methodology.

1. *”What is the current situation of the river system and environment of Valkenburg aan de Geul?”*
2. *”What can be learnt and/or applied from reference projects?”*
3. *”What are possible alternatives for the design of a flood bypass tunnel in Valkenburg aan de Geul?”*
4. *”What flood bypass tunnel design alternative is selected based on the evaluation of the alternatives?”*
5. *”To what extent does the selected design alternative semi-quantitatively reduces the risk of flooding in Valkenburg aan de Geul?”*

1.4. Scope

- The design applies the cost-efficient pipe-jacking method for construction. The method has a maximal inside diameter of 3.5 m. Since the building cost per tunnel remains similar for different diameters while the discharge reduces greatly for a smaller diameter De Leeuw & Mondeel (2021), this diameter is applied in the design. For this diameter two tunnels were estimated to be needed to reduce the flood risk significantly, therefore this was applied in the design as well.
- The thesis is focused on reducing the flood risk in Valkenburg aan de Geul which means that upstream and downstream locations are not incorporated into the design. Although the author is aware that a greater or quicker discharge might have implications for locations further downstream, this is outside the scope of the study.
- The thesis is focused on reducing the flood risk in Valkenburg aan de Geul with a flood bypass tunnel therefore other flood reduction measures are not taken into account. Although the author is aware that a combination of flood reduction measures, such as an additional culvert beneath the Julianakanaal and additional retention areas further upstream and downstream of Valkenburg, would likely be the most effective way to reduce the risk of flooding, this is outside the scope of the study. An overview of the scope is graphically explained in Figure 1.7.

- The thesis is focused on the hydraulic design and operation of the flood bypass tunnel therefore the detailed structural design, broader societal effects and a detailed calculation of the risk reduction will not be taken into account. Although the author is aware that this might change the proposed design. An overview of the scope is graphically explained in Figure 1.6.

	Inlet	Tunnel	Outlet
Operational design	✓	✓	✓
Hydraulic design	✓	✓	✓
Structural design	✕	✕	✕
Maintenance design	✓	✓	✓

Figure 1.6: The scope is focused on the operational, hydraulic and maintenance design

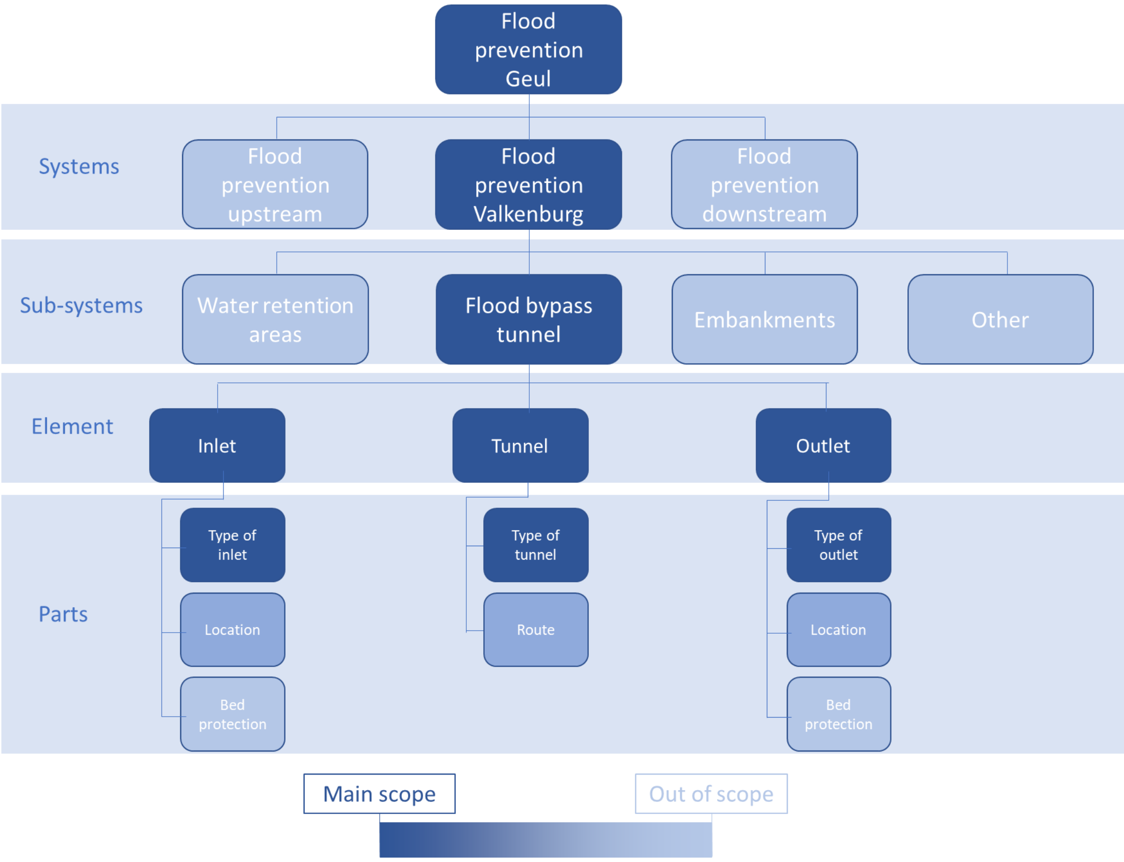


Figure 1.7: The scope is focused on the flood bypass tunnel using Systems Engineering

1.5. Methodology

The approach to reach the objective is to combine a design approach for hydraulic structures with a Systems Engineering approach, where the systems are divided into sub-systems and into elements, see Figure 1.7. Appendix A gives more in-depth information about the various facets of the methodology.

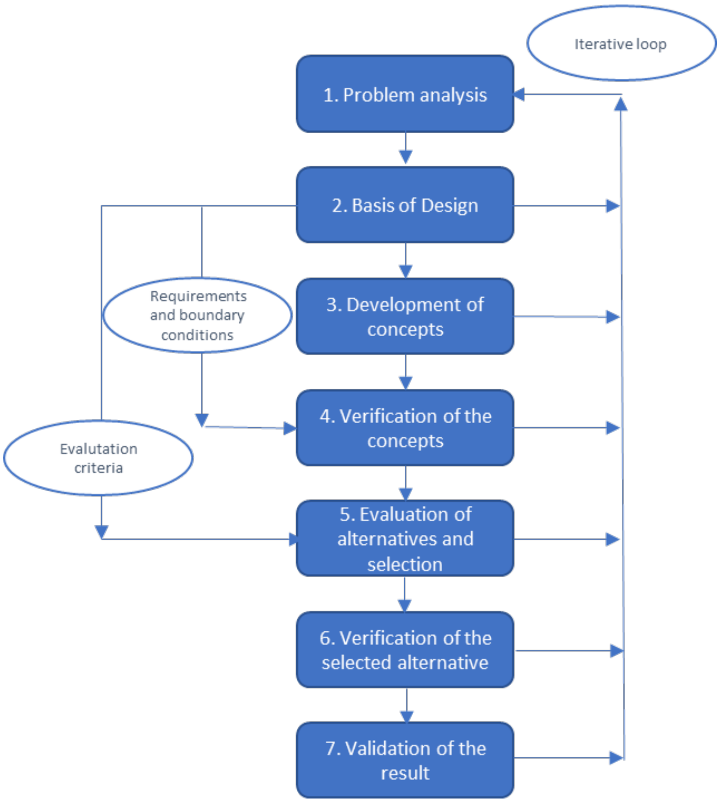


Figure 1.8: Design method for hydraulic structures

Phase 1: Exploration of the problem
In the first phase, the problem is explored by answering the questions: ”who, when and where” using literature. For this stage the following content will be researched:

- Problem analysis;
- System analysis;
- Current situation in non-flooding, flooding and extreme conditions (July 2021);
- Stakeholder-analysis

Phase 2: Basis of Design
In the second phase, the design definition is determined based on the requirements, boundary conditions and evaluation criteria. These are derived from the literature using the problem analysis, area analysis, current situation in non-flooding, flooding and extreme conditions (July 2021) and stakeholder analysis.

- Requirements
- Boundary conditions
- Evaluation criteria

Phase 3: Developments of concepts
The third phase is the development of concepts which will give possible solutions. The development of these concepts is based on the reference projects and the design definition.

- Reference projects
- Different design alternatives for the flood bypass tunnel

Phase 4: Verification of concepts
In the fourth phase, the concepts of phase 2 are verified with the requirements and boundary conditions of phase 2. In addition, hydraulic calculations are performed to verify the concepts.

- Verify boundary conditions and requirements
- Hydraulic calculations to verify the concepts
- Verify with stakeholders, in this case with the waterboard of Limburg

Phase 5: Evaluation of alternatives and selection
In the fifth phase, the concepts are evaluated in a multi-criteria analysis, using the evaluation criteria of phase 2, and a cost analysis is done to select the preferred solution.

- Multi-criteria analysis selecting the alternatives based on the evaluation criteria which are determined with stakeholders, in this case with the waterboard of Limburg and the municipality of Valkenburg aan de Geul
- Cost analysis
- Selection of the preferred solution

Phase 6: Verification of the selected alternative
In the sixth phase, the selected alternative is verified semi-quantitatively on reducing the risk of flooding in Valkenburg aan de Geul.

- Semi-quantitative verification on reducing the risk of flooding in Valkenburg aan de Geul

Phase 7: Validation of the result
In the final phase, the preferred alternative is validated and recommendations will follow.

- Validation of the preferred alternative.

1.6. Report outline

The report's structure adheres to the methodology's phases and the research sub-questions. The current situation of Valkenburg aan de Geul's river system and environment are addressed in Chapter 2. The basis of design, which establishes the starting points, requirements, boundary conditions, and evaluation criteria, is described in Chapter 3. The aspects that can be learnt and/or applied from reference projects are presented in Chapter 4. The flood bypass tunnel design alternatives are presented in Chapter 5. The alternatives are evaluated and selected based on multicriteria analysis and cost analysis in Chapter 6. A semi-quantitative analysis of the reduction of the risk of flooding in Valkenburg aan de Geul is provided in Chapter 7. Finally, Chapter 8 discusses the research and Chapter 9 provides the conclusion and recommendation. This is illustrated in Figure 1.9.

Furthermore, Appendix A elaborates on the methods used to reach the objective. Appendix B provides extra information to the system analysis. Appendix C provides extra information on the Basis of Design of Chapter. Appendix D gives an overview of reference projects. Appendix E provides extra information for the hydraulic design. Appendix F weights the factors of the criteria and evaluates the design alternatives. Appendix G elaborates on the cost analysis. Appendix H considers some add-on opportunities and Appendix I considers some sustainable opportunities.

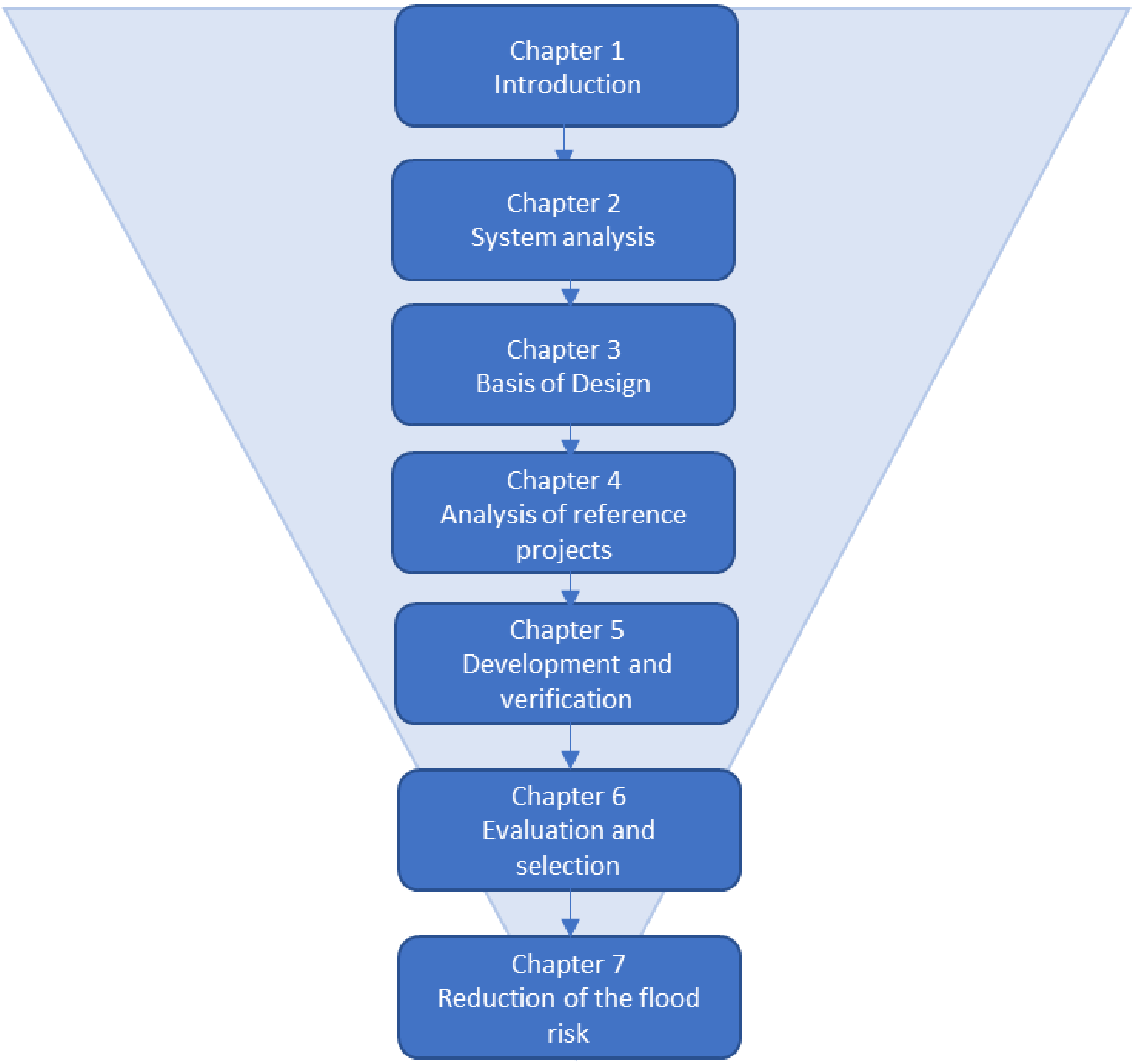


Figure 1.9: Outline report

2 System analysis

The chapter analyses the area, the current Geul system, the discharge conditions and other previous events. The chapter is structured as follows. First, the area is analysed in Section 2.1 after which the current Geul system is analysed in Section 2.2, the discharge conditions are analysed in Section 2.3 and other previous events are analysed in Section 2.4.

2.1. Area analysis

The area analysis is divided into a location in Section 2.1.1 explaining the elevation difference, the cultural heritage and nature of Valkenburg aan de Geul, and a geotechnical analysis in Section 2.1.2 elaborating on the soil layers and groundwater at Valkenburg aan de Geul.

2.1.1. Location analysis

The location of Valkenburg aan de Geul is analysed in this section.

Low-laying Valkenburg aan de Geul

Valkenburg aan de Geul lies in the valley of the river Geul, see Figure 2.1. Due to the small river and lack of space, the water level goes up significantly with a higher flow rate as the valley works as a funnel for the catchment area (Task Force Fact Finding hoogwater 2021, 2021).

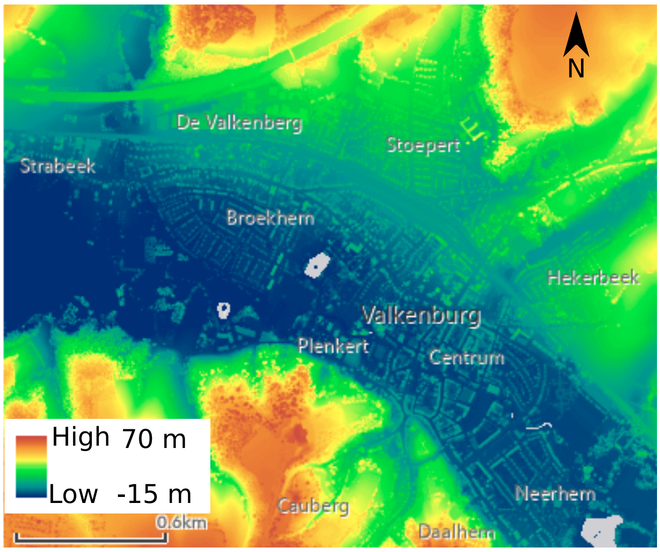


Figure 2.1: Elevation map (Algemeen Hoogtebestand, 2022)

Picturesque historic town in nature reserve
Valkenburg aan de Geul is a cultural heritage-protected picturesque town in a nature reserve (Natura 2000) (Binnenlands bestuur, 2021). Valkenburg has a rich past as a fortified town known for its marl. The town consists of castles, Roman catacombs, marl-caves, city walls and gates (Visit Zuid Limburg, 2022). Together there are almost 200 officially registered national monuments, see Appendix B.1 for an overview of the locations of the monuments.

Natura 2000 is a European collaboration of protected nature reserves. In these areas, endangered plant and animal species are protected by protecting their natural habitat to preserve biodiversity (Natura 2000, 2000). Valkenburg is part of the Geuldal, which is one of the biggest Natura 2000 areas of the Netherlands with the most variety due to the large height differences, which makes it gradient-rich, see Appendix B.1 for the Natura 2000 area of Valkenburg aan de Geul. In the valley, there are relatively nutrient-rich and wet to moist soils with an alternation of meadows and various forest communities. The higher, dry slopes consist of a nutrient-poor and calcareous upper half and a somewhat more nutrient-rich lower half, with limestone sometimes outcropping (in grooves). nutrient-rich and moist soils in the stream valley, which are bordered on both sides by nutrient-poor plateaus higher up. The grasslands and forests that occur here contain orchid-rich hillside forests, calcareous grasslands, hectic grasslands and vegetation on rock edges. The Geuldal is important for basketed and pale bats, as well as stag beetle, yellow-bellied toad and the Spanish flag. Common Kingfisher, Griffon Bat, Thunderbird, Yellow-bellied Toad and Hazel Mouse (Natura 2000, 2000). Due to the city's and nature's beauty, Valkenburg aan the Geul is a busy tourism spot, therefore the city centre has many restaurants and hotels (Binnenlands bestuur, 2021).

In the Natura 2000 area Geuldal, there are many special and diverse habitat types which are sensitive to nitrogen. Therefore measures have been taken to protect the nature reserve from this Provincie Limburg (2017). Due to the measures and measurements of this area, the goals set for the conservation and/or lowering of nitrogen deposition will be achieved Provincie Limburg (2017). Due to the current nitrogen crisis in the Netherlands and its certainty, this part is neglected. However, future research should take this into account.

2.1.2. Geo-technical analysis
The geotechnical analysis is examined in this section.

Soil
The first soil layers, just below ground level until around 3.5 m depth, consists of soft materials such as clay and fine sand. On average followed by varied layers of gravel and limestone (TNO Geologische Dienst Nederland, 2022). The precise measures of DINO are added in Appendix B.3.

Ground functions as a sponge
Bureau Stroming (2022) found out that during the July 2021 event, 80 % of the rain of the Dutch part of the Geul was retained by the ground due to the silt- and gravel soil. Until the end of the rain period (+30 hours), the water was retained in the ground and the 'sponge' was not saturated yet. The effect of the sponge depended however on the land use. High vegetation retains more water, while cornfields and urbanized areas speed up the flow. In forests, 40 % of the rainfall is retained by the leaves. Therefore in total 80 to 85 % of the rainwater falling in the Netherlands did not contribute to the flood. In Belgium, only 50 % of the rain fallen here was retained due to a thin soil layer. Together 50 to 65 % of the rainfall in Belgium was retained (Bureau Stroming, 2022).

2.2. Current system
The Geul originates in Belgium. Here the river runs much steeper, than in the Netherlands which contributes to a quick discharge downstream together with the soil that is impermeable (Paarlberg, 1990). This is typical for smaller catchment areas in mountainous regions. The Geul has a couple of side branches which come together at Gulpen, see Figure 2.2. Water buffers are present upstream of Valkenburg which can hold a large amount of precipitation. Just upstream of Valkenburg, see Appendix B.1 for the inundation area upstream of Valkenburg, the Geulpark and an area near Castle Genhoes function as an inundation area, while being part of the Natura 2000 reserve as well.

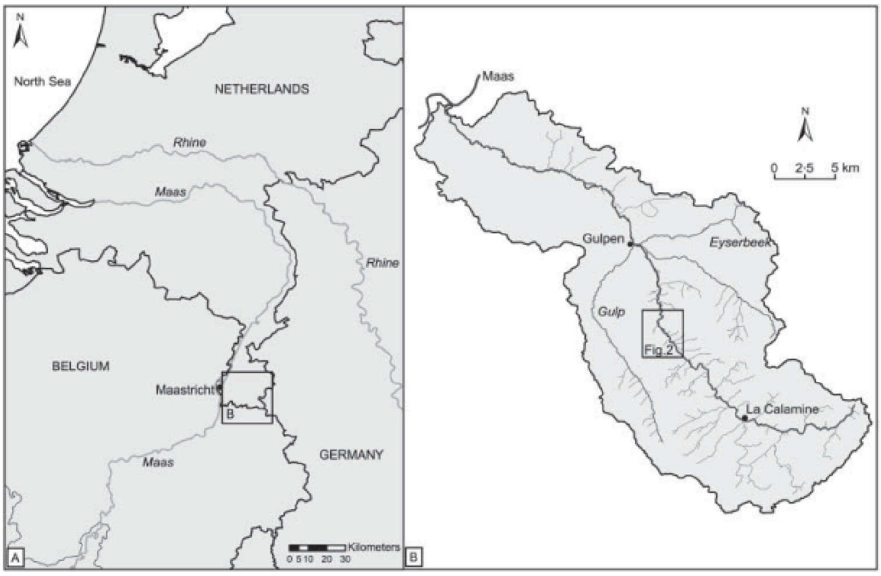


Figure 2.2: Geul catchment area (De Moor et al., 2007)

When the Geul reaches Valkenburg, it bifurcates into two canals, after passing through the historic centre, the canals rejoin, see Figure 2.3. The Walram weir divides the flow between the two canals. During a flood, the discharge capacity through this part of the Geul is limited due to the bridges and watermills that span over the canal (Kallen et al., 2022).

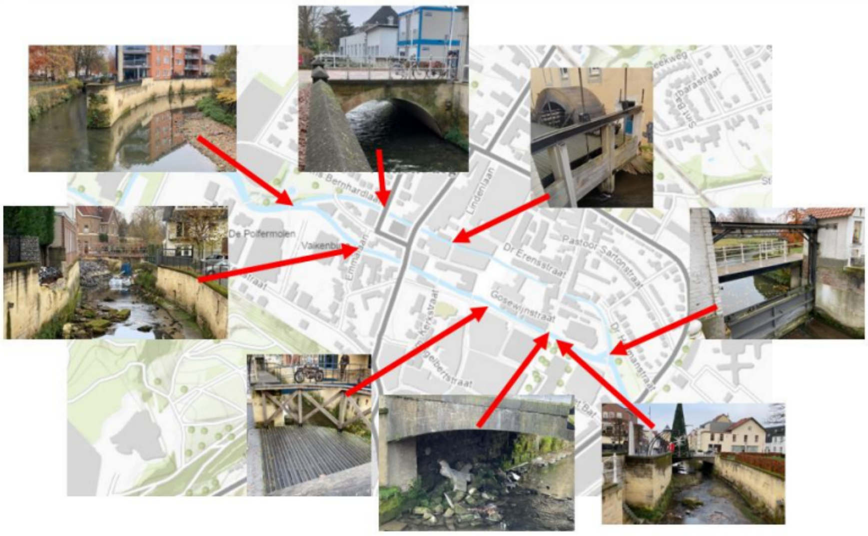


Figure 2.3: Current flood defence measures in Valkenburg (Kallen et al., 2022)

Downstream of Valkenburg, there are some other hydraulic flood defence measures. The biggest one is at the Geulmonding due to the limited discharge through the siphon underneath the Juliana canal together with the high water in the Maas (Asselman et al., 2022), see the report on this part of the Geul by Deltares. However, in 2021, just downstream of Valkenburg, a high-water flood side channel was constructed, see Figure 2.4. This is constructed to improve the discharge during a heavy rain event and to prevent flooding in the city centre of Valkenburg. It is expected to be in function a couple of times a year as it is designed so that no inundation will occur along the Plenkertstraat and reduction of more inundation area during a return period of 25 years (Waterschap Limburg, 2021). During the flood of July 2021, the flood side channel proved its function. Together with this project, the fish migration is improved by flatting the bed with gravel to make it easier for fish to swim here, also the quays are repaired following deferred maintenance (Waterschap Limburg, 2021).

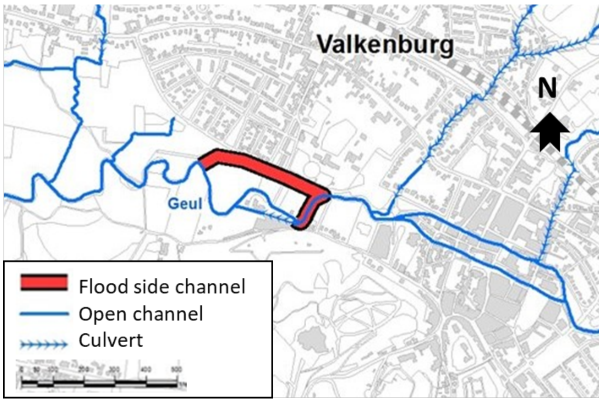


Figure 2.4: Flood side channel downstream of Valkenburg; edited (Waterschap Limburg, 2021)

2.3. Discharge conditions

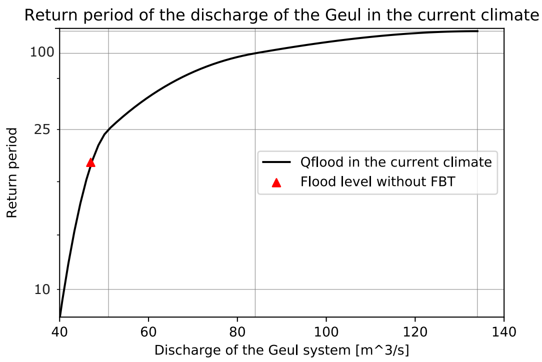
The River Geul in Valkenburg is flooding at a discharge of 47 m^3/s (Deuss et al., 2016). The average discharge of the Geul is 1-4 m³/s (Paarlberg, 1990). The discharges with a flood once every 25 years and 100 years are given in Table 2.1. The difference with the average discharge is enormous due to Geul's dependence on precipitation.

Table 2.1: Discharges along the Geul (van Heeringen et al., 2022)

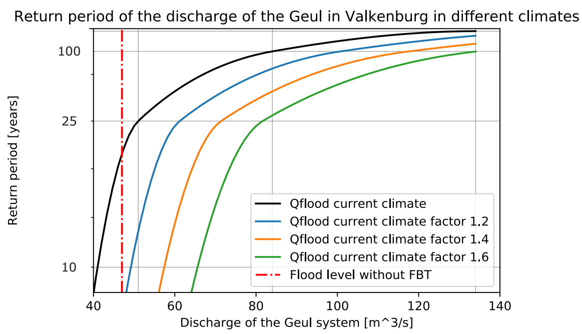
Location | Return-periods	T = **25**	T = **100**	July 2021 T ≈**900**
Valkenburg - Hertenkamp	51 m^3/s	84 m^3/s	134 m^3/s

The return periods with occurring discharges estimated by van Heeringen et al. (2022) are numerically interpolated with a piecewise cubic hermite interpolating polynomial. This interpolation preserves monotonicity in the interpolation data and does not overshoot if the data is not smooth (Moler, 2004). This creates the smooth cumulative probability of occurrence graph see Figure 2.5a.

Climate change increases the probability of a higher discharge, see the sensitivity analysis about the climate change projections in Appendix C.2. The climate change projections shift the curve to a lower return period with the same discharge, see Figure 2.5b.



(a) Return period of the discharge of the Geul in the current climate



(b) Return period of the discharge of the Geul in Valkenburg in different climates

Figure 2.5: Estimated discharges with return periods (PCHIP) one-dimensional monotonic cubic interpolated on a vertical logarithmic scale

Discharges July 2021

The peak discharge of the July 2021 flood was estimated by van Heeringen et al. (2022) using model calculations to be around 134 m^3/s for the Geul at Valkenburg, see Table 2.1. The measured rainfall and discharge have never been this much since precipitation measurements started, especially during the summer. It is estimated that the event will happen once every 100 to 1000 years (Task Force Fact Finding hoogwater 2021, 2021). Following van Heeringen et al. (2022) the occurrence rate of rainfall is once every 900 years. For Valkenburg, this is a bit lower as Valkenburg is located at the end of the flow area. In Figure 2.6 the flooded area of Valkenburg aan de Geul is indicated.

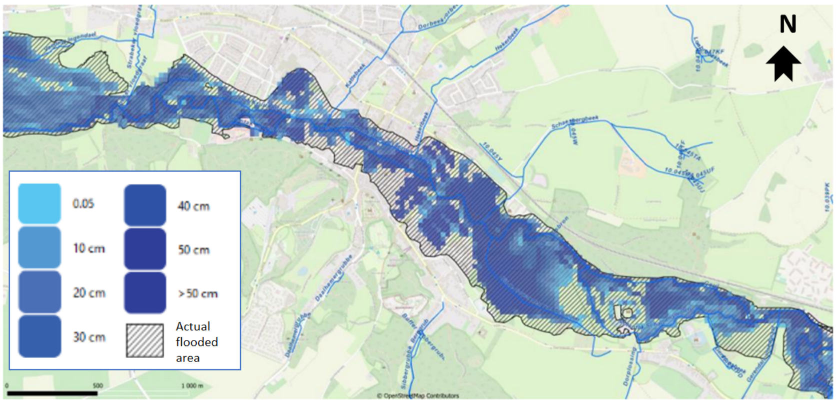


Figure 2.6: Area flooded during the July 2021 flood; edited of (van Heeringen et al., 2022)

2.4. Other previous events

Flood February 2022 - Geul

In February 2022 the river Geul threatened to burst its banks again. The water level was just 25 to 30 centimetres below the top of the bank in Valkenburg. The fire brigade was ready to help with bags of sand and pumps (PZC, 2022). Luckily only 16 people had to be evacuated due to flooding at Epen (upstream of Valkenburg). Most of the water originated in Belgium due to a heavy rainstorm (NU, 2022).

Floods 1993 and 1995 - Meuse

In 1993 and 1995 Limburg was flooded as well. Here most damage and losses were recorded in the main Meuse floodplain, instead of in the regional rivers, as the Geul floodplain. The estimated damage in 1993 and 1995 (converted to 2021 prices) is around € 200 million and € 125 million, respectively, excluding damage due to business interruption. Thus the damage in Valkenburg aan de Geul is larger than for the total floods in 1993 and 1995 (Task Force Fact Finding hoogwater 2021, 2021).

Following these flooding events, many measures have been implemented by Rijkswaterstaat, which reduced the damages along the Meuse during the floods of 2021. The measures taken are broadening and deepening the Meuse (Joost Schreurs, 2021). This was done using the 'room for the river' principle, as side channels and flood planes were dug out as well as the quays and dikes are strengthened to meet the requirements for primary barriers in 2050 (Hoogwaterbeschermingsprogramma) (Rijkswaterstaat, 2020). As the Meuse is a big river this is authorized by Rijkswaterstaat. The Geul is a stream, thus authorized by Waterschap Limburg (Joost Schreurs, 2021).

Wet and dry summers

The summer of 2014 and 2016 was very wet as well, but Limburg also experienced drought. In the summer of 2020, there was an extreme drought, just as in the summers of 1947, 1976, 1959, 2011, and 2018 (Jos van den Broek, 2020). The measures to keep Limburg from drought is to retain water in the summer, which means closing all the weirs. Therefore the waterboard is extra alert once this draught turns into a wet period, as the water has to flow downstream to prevent floodings (Waterschap Limburg, 2022a).

3

Basis of Design

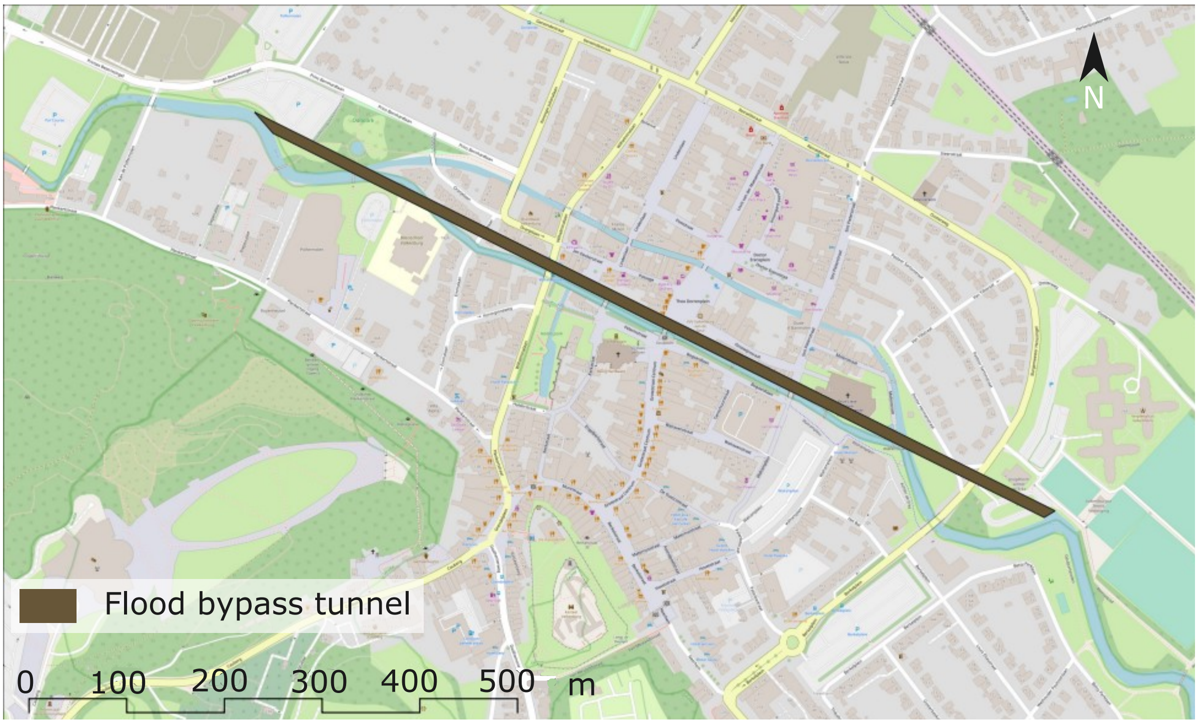
This chapter describes the status quo, requirements, boundary conditions and evaluation criteria of the hydraulic design. First, the status quo is discussed in Section 3.1, followed by the requirements and boundary conditions in Section 3.2 and in Section 3.3 respectively. The evaluation criteria are briefly discussed in Section 3.4.

3.1. Status quo

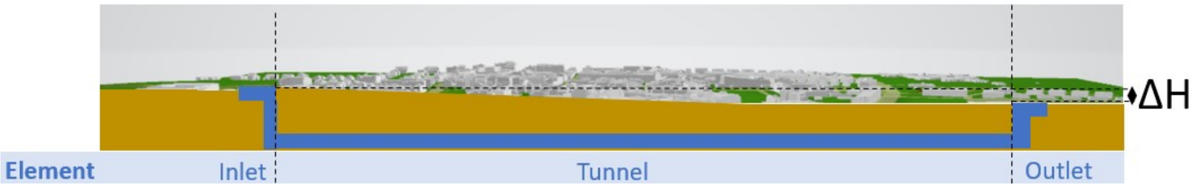
The status quo of the design is described by the tunnel trajectory and the location of the in- and outlet. This is discussed respectively in Section 3.1.1 and in Section 3.1.2.

3.1.1. Tunnel trajectory

The location of the tunnel is proposed by De Leeuw & Mondeel (2021). The tunnel will lay at its maximum depth of approximately 10 m under ground level. The two tubes will be bored next to each other. This way the borings are far underneath the houses and will not interfere with other installations in the ground and foundations of houses, nonetheless the track is preferred to deviate slightly from Figure 3.1 to fit underneath the Geul branch to ensure the tunnel will not cross properties, this is discussed in Chapter 8. The bends are very smooth so no additional friction losses are expected. The tunnel will be approximately 875m long with a total horizontal length of 850 m. From Figure 1.5 it follows that an approximately 2.65 m difference in head is applied.



(a) Location the tunnel trajectory (QGIS, n.d.)



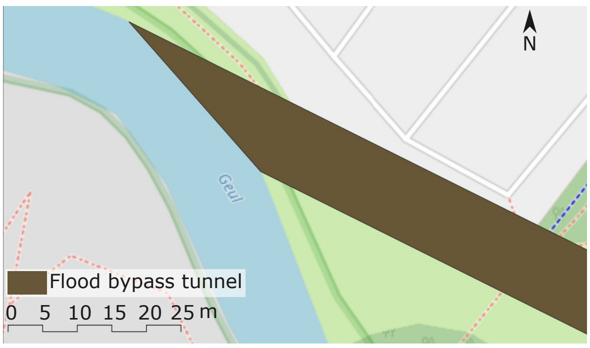
(b) Location the tunnel trajectory cross section *schematized*

Figure 3.1: Tunnel trajectory

3.1.2. Location in- and outlet

The location of the inlet is proposed by De Leeuw & Mondeel (2021). The outlet location however is based on a combination of the proposed outlet location of De Leeuw & Mondeel (2021) and the outlet by Leijser & Nijhof (2022). The result is an outlet location where the tunnel smoothly rejoins the Geul, promoting straight streamlines and reducing the number of bends in the tunnel, while the building pit is still located at a parking site. See the analysis in Appendix C.1. In Figure 3.2 the in- and outlet locations are shown.

(a) Connection of the system to the inlet



(b) Connection of outlet to the system

Figure 3.2: Connections of the in- and outlet to the system

3.2. Boundary conditions

The boundary conditions of the design are the geotechnical boundary conditions, constructional boundary conditions, discharge conditions and water-level conditions. These are discussed respectively in Section 3.2.1, Section 3.2.2, Section 3.2.3 and Section 3.2.4.

3.2.1. Geo-technical boundary conditions

The first soil layers, just below ground level until around 3.5 m depth, consist of a variation of sandy clay, medium and fine sand, clay and peat and a little coarse sand. On average followed by a limestone layer, consisting of limestone with a few flint banks. (TNO Geologische Dienst Nederland, 2022). The measures of DINO Loket are added in Appendix B.3. See Figure 3.3 for the estimated soil layers. For more reliable data, more measures need to be taken to ensure the tunnel will be bored through the limestone.

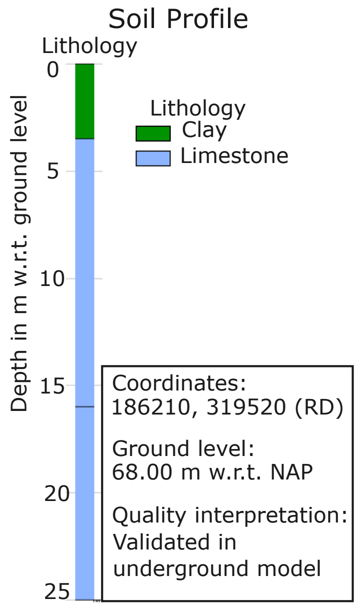


Figure 3.3: Soil profile; based on (TNO Geologische Dienst Nederland, 2022)

3.2.2. Constructional boundary conditions

Boring a tunnel using a drilling method is very expensive due to the drilling equipment needed. The pipe jacking method costs less due to the technique. The pipe is forced into the ground, which requires less expensive equipment. This can be seen when comparing Van Dijk (2022)'s drilling tunnel with De Leeuw & Mondeel (2021)'s pipe jacking tunnel, Table 3.1. This makes this technique for this application cost-efficient. In addition, it is frequently used for sewage and freshwater pipes in the Netherlands. However, only smaller diameters are applicable with this technique with a maximal inside diameter of approximately 3.5 m. Even if this would require multiple tubes for Valkenburg, it would still be more cost-efficient compared to the boring method. The diameter of 3.5 m would be optimal in cost since the building cost per tunnel remains similar for different diameters while the discharge reduces greatly for a smaller diameter De Leeuw & Mondeel (2021). The pipe jacking technique is applicable in almost all soils, including limestone, however, rough gravel should be avoided. The construction process of the pipe jacking method requires a deep construction pit of 15 meters long and 10 meters deep at both the in- and outlet to pipe jack the concrete tubes. An in-depth analysis of this can be found in Appendix C.2. Due to the benefits of the pipe jacking construction method, the design alternatives are all based on this, thus they have a maximum diameter of 3.5 m and are constructed using two shafts. Due to the small difference in head (approximately 2.65 m) and the small diameter of the tunnel (max 3.5 m), the tunnel needs to be pressurised to reduce friction and prevent high-pressure peaks, which makes the application more predictable.

Table 3.1: Comparing Van Dijk (2022)'s drilling tunnel with De Leeuw & Mondeel (2021)'s pipe jacking tunnel

	Drilled tunnel	2 pipe jacked tunnels
Estimated costs [x million]	112-207 €	29 €
Diameter of the tunnel [m]	8	3
Length [m]	720	800
Discharge [m^3/s]	87	40

3.2.3. Discharge conditions

Non-flooding situation

The average discharge of the Geul is 1-4 m^3/s (Paarlberg, 1990).

Flood situation

According to Deuss et al. (2016), the river Geul is over banking and flooding Valkenburg at a discharge of 47 m^3/s. From Figure 3.4 it follows that the return period corresponding to this flooding discharge is approximately once every 19 years. To prevent the Geul from flooding Valkenburg, an estimate of the needed discharge through the tunnel is calculated using a simplified equation, see Equation 3.1. The calculation is performed under several return period scenarios.

$$Q_{FBT} = Q_{flood} - Q_{Valkenburg} \tag{3.1}$$

where:
Q_{FBT} [m^3/s] = discharge capacity needed through the tunnel
Q_{flood} [m^3/s] = discharge of the flood with an estimate return period
$Q_{Valkenburg}$ [m^3/s] = discharge capacity of the Geul in Valkenburg (= 47 m^3/s (Deuss et al., 2016))

July 2021 flood

The flood bypass tunnel should have a minimum discharge capacity of 87 m^3/s to have mitigated the July 2021 flood discharge.

$$Q_{FBT} = Q_{T\approx 900} - Q_{Valkenburg} = 134 - 47 = 87m^3/s$$

Current safety standard once every 25 years

According to the assumptions used, the flood bypass tunnel should have a minimum discharge capacity of 4 m^3/s in order to meet the current safety standard of once every 25 years.

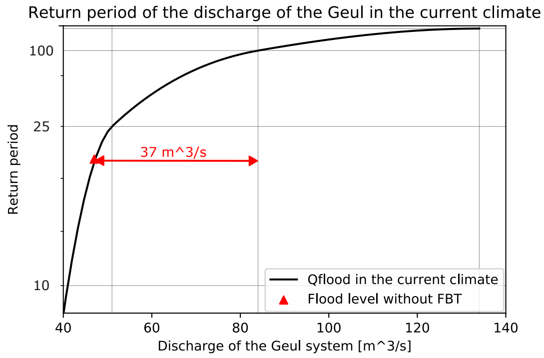
$$Q_{FBT} = Q_{T=25} - Q_{Valkenburg} = 51 - 47 = 4m^3/s$$

The deep uncertainty regarding climate change, mentioned in Section 1.2.2, together with the lifetime requirement of the flood bypass tunnel of at least 100 years, see Section 3.3.3, will make the discharge corresponding with the current return period shift, see the sensitivity analysis about the climate change projections in Appendix C.2. The climate change projections shift the curve to a lower return period with the same discharge, see Figure 3.4b. The discharge corresponding with the future 25-year return period switches towards the current 100-year return period. Therefore this return period is used to buffer the uncertainty of climate change.

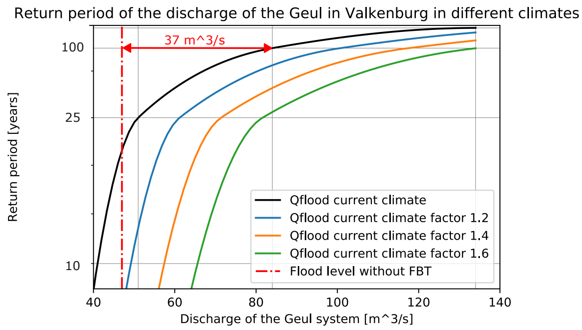
Safety standard once every 100 years

The flood bypass tunnel should have a minimum discharge capacity of 37 m^3/s in order to meet the safety standard of 1/100 years. This is the hydraulic requirement for the design of the flood bypass tunnel. See Figure 3.4 for a graphic explanation.

$$Q_{FBT} = Q_{T=100} - Q_{Valkenburg} = 84 - 47 = 37m^3/s$$



(a) Return period of the discharge of the Geul in the current climate

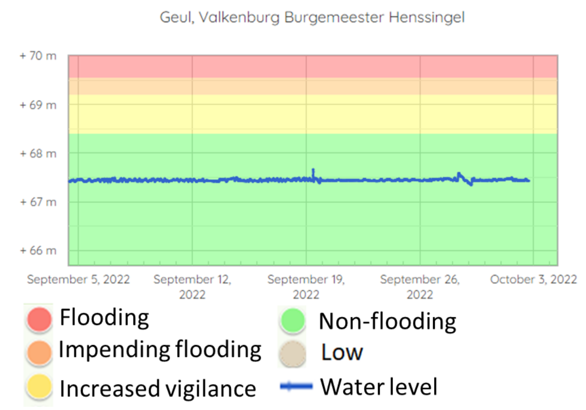


(b) Return period of the discharge of the Geul in Valkenburg in different climates

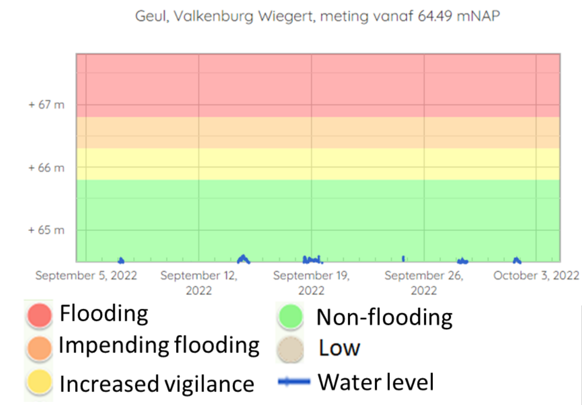
Figure 3.4: Estimated discharges with return periods (PCHIP) one-dimensional monotonic cubic interpolated on a vertical logarithmic scale

3.2.4. Water-level conditions

Near the inlet and outlet locations, the water level is measured by Waterschap Limburg (2022b), these are used as reference points for the in- and outlet location, see Figure 3.5 and Figure 3.6



(a) Waterlevel Burgemeester Henssingel(Waterschap Limburg, 2022b)



(b) Waterlevel Wiegert (Waterschap Limburg, 2022b)

Figure 3.5: Waterlevels

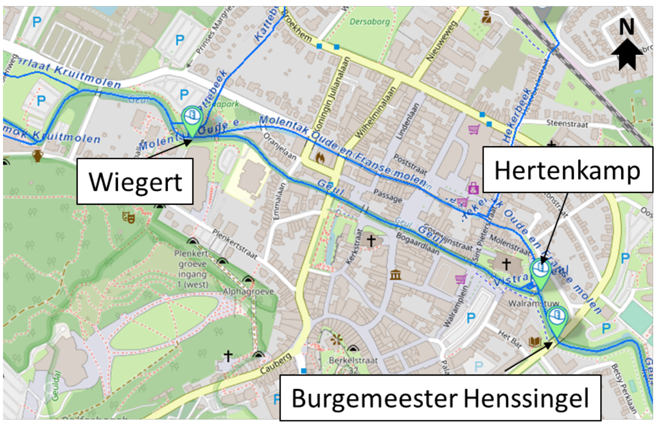


Figure 3.6: Locations of recorded waterlevels(Waterschap Limburg, 2022b)

Non-flooding situation
At the location of the inlet, the water level will be in a non-flooding situation up until CD + 68.4 m, see Figure 3.5a. According to Figure 1.5 the bottom of the river is located at CD + 66 m. This makes a water depth in a non-flooding situation 0 to 2.4 m, due to the Geul's dependence on precipitation and the average discharge of 1-4 m^3/s (Paarlberg, 1990) the average water depth will lay much lower than 2.4 m.

At the location of the outlet, the water level will be in a non-flooding situation up until CD + 65.75 m, see Figure 3.5b. According to Figure 1.5 the bottom of the river is located at CD + 63.35 m. This makes the water depth in a non-flooding situation also 0 to 2.4 m. Again due to the Geul's dependence on precipitation, the average water depth will lay much lower than 2.4 m.

Flood situation
At the location of the inlet, the water level during a flood will be equal to or larger than CD + 69.5 m, see Figure 3.5a. This correspondent with a 3.5-meter flood water depth.

At the location of the outlet, the water level will be equal to or larger than CD + 66.85 m, see Figure 3.5b. This correspondent with a 3.5-meter flood water depth.

3.3. Requirements

The requirements of the design are the hydraulic requirements, operational and maintenance requirements and lifetime requirements. These are discussed respectively in Section 3.3.1, Section 3.3.2 and Section 3.3.3.

3.3.1. Hydraulic requirements

The hydraulic requirements are a result of the boundary conditions and conclusions drawn from the reference projects.

1. The function of the flood water bypass tunnel is to divert the water through the tunnel during a flood. Therefore during a flood event with a return period of 100 years, the tunnel has to divert at least 37 m^3/s, see the boundary conditions. Based on the graph of return periods 3.4a the operational return period of the tunnel is at least once every 19 years, as without the tunnel Valkenburg would flood with this return period. Nonetheless, the tunnel is in operation more often than this to reduce the risk of such a flood occurring by functioning during increased vigilance water level as stated by the colour code of Waterschap Limburg (2022b).
2. Due to the pipe jacking method restriction the tunnel has a maximum diameter of 3.5 m.
3. The pressurised flow decreases friction and prevents pressure peaks.

3.3.2. Operational and maintenance requirements

The operational and maintenance requirements are a result of the system analysis and suggestions by the municipality and waterboard.

1. The intake has to be regulated to make sure that in no flood condition, the water will run through the Geul instead of the tunnel and at the beginning of a flood the water will also run controlled through both the tunnel and the Geul to avoid sudden floods and pressure peaks.
2. High reliability. The tunnel should be able to function whenever needed. During a non-flooding situation, the water should flow through Valkenburg city, to ensure the Geul will keep its beauty in the city centre.
3. The tunnel should be safe for the public, by avoiding the possibility of entering the tunnel by accident and maintaining the water quality.
4. Debris and others should not interfere with the operation of the tunnel.
5. In order to provide maintenance, inspection, and repairs, all parts of the tunnel including the in- and outlet must be accessible in a safe manner.

3.3.3. Lifetime requirements

The structural lifetime should be at least 100 years, while the electrical and mechanical lifetime should be 20 and 50 years respectively (M. Z. Voorendt & Molenaar, 2019).

Table 3.2: Lifetime requirements

	Technical Lifetime
Structural	100 years
Electrical & ICT	15 years
Mechanical	25 years

3.4. Evaluation criteria

The evaluation criteria for the multi-criteria analysis are based on discussions with the municipality and waterboard. The elaboration for each criterion is explained in Chapter 6, here a short overview is given.

1. **Effectiveness**;
   The effectiveness of the flood bypass tunnel is determined by the discharge capacity of the flood bypass tunnel during design operation.
2. **Reliability**;
   The reliability is determined by the number of steps that have to be taken during the operation, which makes the system more complex. As more steps have a large chance of failure.
3. **Control**;
   The system's control is determined by the degree of regulation over the discharge that flows through the flood bypass tunnel during operation.
4. **Safety**;
   Safety is determined by the limitation of access of civilians to the flood bypass tunnel. The danger of falling in, especially for children and dogs etc., should be avoided.
5. **Maintainability**;
   Maintenance ensures that the flood bypass tunnel is operational during a flood. Therefore the structure must be easy to maintain with good access for possible inspections and possible replacements.
6. **Effect on the environment**;
   The effect on the environment is determined by the structure's integration into its surroundings, and the impact the structure has on the environment in terms of nature-friendliness

4 Analysis of reference projects

This chapter compares reference flood bypass tunnel projects in Switzerland and the USA and reference side channel projects in the Netherlands to the proposed flood bypass tunnel of Valkenburg to deduce what characteristics could be applicable. The comparison is based on the characteristics of the reference projects which are elaborated in Appendix B and on the proposed flood bypass tunnel of Valkenburg resulting from the system analysis in Chapter 2 and the design definition in Chapter 4. The reference projects are analysed for their applicability to Valkenburg. The results are discussed and summarised.

The chapter is structured as follows. First, the applicability of the flood bypass reference projects is discussed in Section 4.1, after which the applicability of the side-channel reference projects is discussed in Section 4.2. This is followed by the summary in Section 4.3.

4.1. Flood bypass tunnel reference projects

The main function of a flood bypass tunnel is to rapidly convey floodwater around the constrained area. The flood bypass tunnel is commonly built in densely populated areas, has a short length due to its cost, and is often constructed from stone, concrete, or metal. Compared to other bypass solutions they do not retain water and take up the smallest area of land since other structures can be placed on the ground above the flood bypass tunnel(Serra Llobet et al., 2021).

There are many different flood bypass tunnels applied in mostly Switzerland and the USA. The considered reference projects are the flood bypass tunnels of Lyss, Sarneraa, Thun, Dallas, and San Antonio. These projects were chosen because of their differences in characteristics, however, there are of course many more examples than these. The reference projects are discussed individually, comparing the applicability of important features to the proposed flood bypass tunnel in Valkenburg. These project-specific characteristics are summarised in Table 4.1, and the (non-)applicable features for each project in comparison to the proposed Valkenburg flood bypass tunnel are detailed in Table 4.2. The extensive description and figures of the various reference projects can be found in Appendix B.

Table 4.1: Table summarising the characteristics of the five flood bypass tunnel reference projects

	Lyss	**Sarneraa**	**Thun**	**Dallas**	**San Antonio**	*Valkenburg*
Active or Passive operation	Active	Active	Active	Passive	Passive	
Operation since	2012	2025	2012	2023	1996	
Pressurised tunnel	x	✓	✓	✓	✓	✓
Construction method	Drilling	Drilling	Drilling	Drilling	Drilling	*pipe jacking*
Cost at construction year [x million]	48 CHF	200 CHF	62 CHF	320 $	111 $	*40 €*
Inside diameter [m]	4	Unk.	5.5	9.1	7.3	*2 x 3.5*
Discharge [m^3/s]	65.8	100	115	424	189	*40*
Length [m]	2570	6500	1129	8100	4900	*850*

Flood bypass tunnel Lyss, Switzerland

Just like the proposed flood bypass tunnel in Valkenburg, the flood bypass tunnel of Lyss runs underneath a town and bifurcates and confluences the river. The flood bypass tunnel is actively operated at the inlet which has a vertical moving flat gate and a sediment basin, to prevent sediment entering and silting up the flood bypass tunnel. The inlet also has a trash rack. The flat gates and trash rack could be applicable for the flood bypass tunnel of Valkenburg as a gate ensures control and a trash rack prevents trash from entering and blocking the flood bypass tunnel. The outlet has a stilling basin to reduce energy before re-entering the channel. This will not be needed for the proposed flood bypass tunnel due to the limited energy available for the Valkenburg flood bypass tunnel compared to Lyss's. However, the drilled tunnel of 4 m diameter will not be applicable due to its high costs, therefore the big diameter is not applicable and neither is the non-pressurised tunnel which reduces the discharge and is prone to pressure peaks. The pressure peaks could increase the maintenance costs, as it may lead to quicker or more erosion of the tunnel.

Flood bypass tunnel Sarneraa, Switzerland

Unlike the proposed flood bypass tunnel in Valkenburg, the flood bypass tunnel in Sarneraa discharges from a lake to a river. The flood bypass tunnel is pressurised with a ventilation shaft which could not be applicable in Valkenburg as the pipe jacking method does not allow for a diversion. The flood bypass tunnel is actively operated by a vertically moving flat gate at both the inlet and the outlet, which could be applicable for Valkenburg. However, the stilling basin at the outlet will not be needed due to the limited energy available for the Valkenburg flood bypass tunnel compared to Sarneraa's. Also, the operation of the flood bypass tunnel could be applied; the Sarneraa flood bypass tunnel is always filled and closed off when not in operation. However, the boring method will not be applicable due to its high costs.

Flood bypass tunnel Thun, Switzerland

Thun is closest related to the proposed flood bypass tunnel in Valkenburg in terms of the discharge as well as the length. Just as in Valkenburg the flood bypass tunnel of Thun runs underneath a town and bifurcates and confluences the river. However, the flood bypass tunnel in Thun is a drilled tunnel with a diameter of 5.5 m, due to the pipe jacking method this will not be applicable. The flood bypass tunnel is actively operated by vertically moving flat gates. The inlet has a flat gate and trash rack that could be applicable for the flood bypass tunnel of Valkenburg as a flat gate ensures control and a trash rack prevents trash from entering and blocking the flood bypass tunnel. The flood bypass tunnel also has a system for early automatic detection of flood hazards, thus it can already operate before the flood, which could be applicable to the proposed flood bypass tunnel. The flood bypass tunnel is filled with water even when it is not in operation, in which case it is closed off. Except for maintenance when it will be drained using pumps. The maintenance could be a great example for the proposed flood bypass tunnel of Valkenburg.

Flood bypass tunnel Dallas, Texas

The flood bypass tunnel of Dallas has an enormous size and discharge, therefore this flood bypass tunnel is barely comparable to the proposed flood bypass tunnel in Valkenburg. The passively operated flood bypass tunnel has multiple drop shafts as inlets and outlet. Due to Valkenburg's small difference in head and small size this can not be applied, as drop shafts will lose too much energy to function optimally. However, emptying the tunnel after the operation could be applicable to the proposed flood bypass tunnel.

Flood bypass tunnel San Antonio, Texas

The flood bypass tunnel of San Antonio also has an enormous size and discharge which differs from the proposed flood bypass tunnel in Valkenburg due to Valkenburg's small difference in head and small size compared to the 10.5 m difference in head of the San Antonio flood bypass tunnel. However, the passively operated inlet of an Ogee weir and trash racks could be applied to regulate the intake in the flood bypass tunnel. The outlet has a drop shaft and stilling basin, which is not applicable to the proposed flood bypass tunnel due to the energy loss. In the non-flooding situation, the water remains in the flood bypass tunnel. The water is disinfected and re-circulated with pumps for water quality enhancement. This is not preferred in Valkenburg due to its costs in comparison with the small size of the flood bypass tunnel.

Table 4.2: Table summarising the applicability of the reference projects to the proposed project in Valkenburg aan de Geul.

		Applicable to flood bypass tunnel Valkenburg	**Not applicable to flood bypass tunnel Valkenburg**
Lyss	Inlet	Flat gate & trash rack	
(Active)	Outlet		Stilling basin
	Operation		Non-pressurised tunnel
Sarneraa	Inlet	Flat gate & trash rack	
(Active)	Outlet	Flat gate	Stilling basin
	Operation	Tunnel always full and closed off when not in operation	Ventilation shaft
Thun	Inlet	Flat gate & trash rack	
(Active)	Outlet	Flat gate	
	Operation	System for early automatic detection of flood hazards Tunnel always full and closed off when not in operation Before maintenance, the tunnel is emptied	
Dallas	Inlet		Drop shafts
(Passive)	Outlet		Drop shaft
	Operation	After operation, the tunnel is emptied	Multiple intakes
San Antonio	Inlet	Ogee weir & trash rack	
(Passive)	Outlet		Drop shaft & stilling basin
	Operation		Re-circulation of the remaining water in the tunnel

4.2. Flood side channel reference projects Netherlands

Flood side channels are secondary channels that are connected to the main channel of a river but are in general much smaller and convey much less discharge than the main channel. Flood side channels increase the discharge capacity of the river, and hence, reduce flood water levels. Unlike flood bypass tunnels, flood side channels or open channel bypasses are frequently applied in the Netherlands. Even though the space in Valkenburg is too limited for this solution the way the flood water enters and exits the flood side channels could be an example of how the flood water could flow into and out of the flood bypass tunnel. Therefore these reference projects are used to see the applicability of an inlet and an outlet which are only operating during floods.

The reference projects looked at are the flood side channels of Roermond and Nijmegen. They were chosen because of their differences in characteristics as one of them is controlled by a gate and the other one uses a weir, however, there are of course many more examples. The reference projects are discussed individually, comparing the applicability of important features to the proposed inlet and outlet of the flood bypass tunnel in Valkenburg. These project-specific characteristics are summarised in Table 4.3. The extensive description and figures of the various reference projects can be found in Appendix B.

Table 4.3: Summary of the flood side channel reference projects

	Groene river, Roermond	**Spiegel waal, Nijmegen**
Active or passive operation	Active	Passive
Cost at construction year [x million]	15 f	358 €
Lowering of the flood water level	Unk.	34 cm
Length [m]	500	4000

Flood side channel Groene rivier, Roermond

The Groene rivier was designed to keep Roermond dry. At high water in the Meuse, the Roer is closed at the mouth and upstream to prevent the Meuse water from flooding the centre of Roermond. The Roerwater is then discharged using a gate to the Groene river. The controlled gate could be applied as an inlet for the Valkenburg flood bypass tunnel. However, it is important to keep in mind that during the summer flood of 2021, the water discharge in the Roer rose as well, therefore the Water Board decided not to activate the system to prevent a dyke breach along the Groene Rivier.

Flood side channel Spiegelwaal, Nijmegen

Nijmegen was a bottleneck for water safety but due to the Spiegelwaal, the water level drops significantly during high water. The inlet is a small weir which overflows during high water and thus fills up the Spiegelwaal, which could be applied as an inlet for the Valkenburg flood bypass tunnel.

4.3. Summary

Most of the reference flood bypass tunnels are large drilled tunnels with a diameter of 4 meters or more. In this way, the reference projects differ from the proposed flood bypass tunnel in Valkenburg as the pipe jacking method allows for a maximum diameter of 3.5 meters. The combination of the pipe jacking method and the small difference in head makes the proposed Valkenburg tunnel unique. Especially in the Netherlands, where this type of structure has never been constructed. Additionally, the pipe jacking method does not enable the construction of a ventilation shaft, therefore this could not be applicable to this project. Furthermore, due to the small difference in head (approximately 2.65 m) and the small diameter of the tunnel (max 3.5 m), the tunnel needs to be pressurised to reduce friction and prevent high-pressure peaks. As a result, energy should not be dissipated, which makes the drop structures and stilling basins not applicable.

Nevertheless, the reference projects are relevant due to their similarity in shape, and application to minimise energy loss and pressure peaks within the pressurised flow, during operation. Although the inlet, outlet, and operation are different for the reference projects, this variation is mostly based on the two types of flood bypass tunnels: a passive and an active, see Table 4.2. The applicable characteristics of these two types of reference projects can be combined into two different flood bypass tunnels:

1. The passive flood bypass tunnel (Dallas and San Antonio) will consist of an Ogee weir and trash rack inlet. After operating, the water in the flood bypass tunnel will be pumped out. The outlet does not follow from the reference projects and will be designed in Chapter 5.
2. The active flood bypass tunnel (Lyss, Sarneraa and Thun) consists of a vertical moving flat gate and a trash rack at the inlet. At the outlet, another vertical moving flat gate will be placed. For the operation, a system for early automatic detection of flood hazards will be applied. Furthermore, the tunnel will always be full and closed off when not in operation. However, for maintenance, the water in the tunnel is pumped out.

5

Development and verification of the flood bypass tunnel design alternatives

Following the analysis of the reference projects, two different flood bypass tunnels are being investigated: a passive flood bypass tunnel and an active flood bypass tunnel. These flood bypass tunnels are developed iteratively, resulting in two flood bypass tunnel design alternatives which meet the requirements described in Chapter 3.

Various previously developed concepts can be used for future research, these are detailed in Appendix H, e.g., retention area upstream, adding barriers in the Geul, using sewage pipe, and usage of different types of gates. In Appendix I sustainable opportunities have been looked at as well. However, they are outside the scope of this work, as the focus is on the hydraulic design and operation of the flood bypass tunnel itself.

This Chapter elaborates on two flood bypass tunnel design alternatives by first explaining the operation and maintenance after which the hydraulic design can be conducted which leads to the design alternative for a passive flood bypass tunnel in Section 5.1, and an active flood bypass tunnel in Section 5.2.

5.1. Design Alternative 1: passive flood bypass tunnel

The design alternative is iterated to find a working combination of the operation and maintenance factors while minimising head losses and preventing air from entering the tunnel to avoid pressure peaks. This design alternative is inspired by the passive inlet of the reference projects of the tunnel of San Antonio's Ogee weir and the side channel of the Spiegel Waal in Nijmegen. The first design alternative is based on a passive inlet, thus does not require a moving object to control the water level but will do so on its own when the water level exceeds the height of the weir. This reduces the probability of failure and decreases maintenance costs of the movable parts, which require mechanical, electrical and ICT elements.

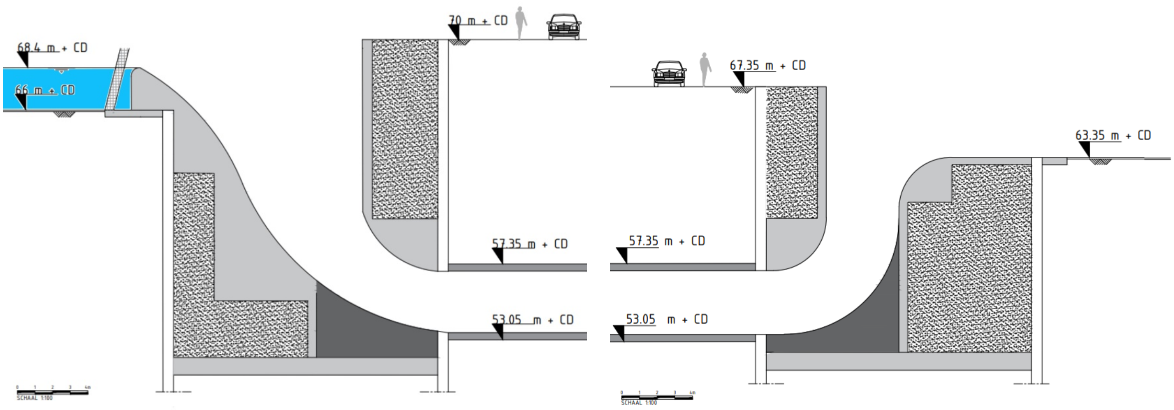
The operation of the design alternative is first explained in Section 5.1.1, after which the design alternative is hydraulically designed in Section 5.1.2 which leads to the final design alternative in Section 5.1.3.

5.1.1. Operation and maintenance of the tunnel

This section describes the operation and maintenance of the tunnel in various situations: non-flooding, flood, after flood, and maintenance situation.

Non-flooding situation

Normally, neither the inlet nor the outlet of the flood bypass tunnel will allow water to enter, see Figure 5.1. The water will only flow through the Geul until the water level of Chart Datum (CD) + 68.40 m at the inlet or CD + 65.75m at the outlet is reached. These water levels are determined using the colour code of Waterschap Limburg (2022b), as described in Section 3.2.4.



(a) Inlet in non-flooding situation

(b) Outlet in non-flooding situation

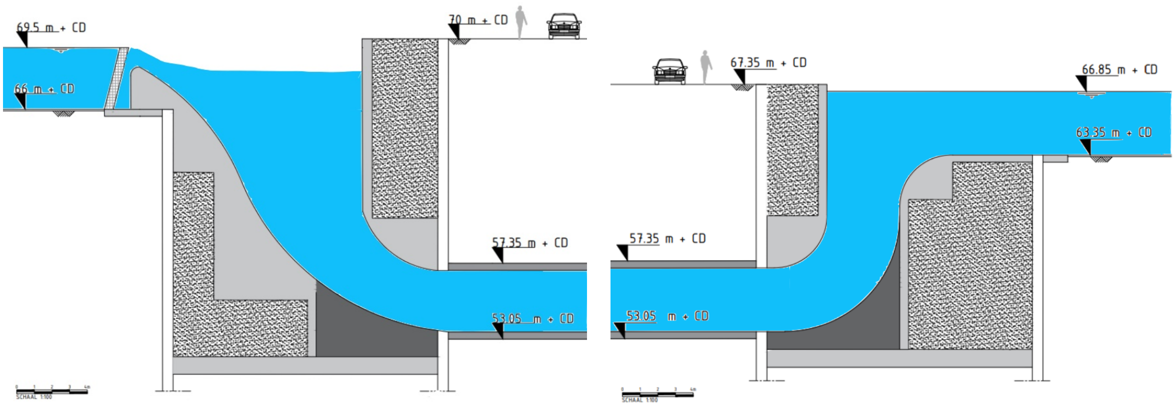
Figure 5.1: Design alternative 1 in the non-flooding situations

Flood situation

When the water level of the Geul at the inlet exceeds the water level in the non-flooding situation of CD + 68.40 m, the water flows through the angled trash rack and over the Ogee weir. The trash rack prevents debris from entering the tunnel and averts blockages as debris floats to the top of the trash rack. Due to the shape of the Ogee weir it effectively discharges water, this is explained in greater detail in Section 5.1.2. For optimal operation of the weir, the flow over the weir should be free-flowing and the weir should not be submerged as this will reduce the discharge, see the hydraulic design in Section 5.1.2.

As the water fills the tunnel and the shafts, the process inevitability causes air to enter the tunnel as well. This needs to be removed to prevent pressure peaks and reduce the discharge capacity of the flood bypass tunnel. The horizontal position of the tunnel will eventually remove this air as it promotes total transport of the air bubbles with the flow for the calculated flow number towards the outflow structure, see the hydraulic design of the tunnel in Section 5.1.2.

Once the tunnel is filled, the horizontal part of the tunnel is submerged which prevents strong air core vortexes to enter the tunnel, this is hydraulically designed in Section 5.1.2 about the inlet. The difference in head which occurs in this stage between the water level in the inlet shaft and outlet shaft discharges the water through the tunnel to the co-current channel (Dutch: meestroomkanaal), see the hydraulic design of the outlet in Section 5.1.2.



(a) Inlet in flood situation **(b)** Outlet in flood situation

Figure 5.2: Design alternative 1 in the flood situations

After flood situation

Due to the passive character of the tunnel, there are no moving parts that close off the tunnel, therefore the water has to be pumped out to ensure the water quality does not degrade while standing still inside the tunnel. To this end, a temporary pump can be installed at the inlet between the two tubes, this will be the lowest point of the tunnel. The tunnel can be emptied from this side once the water level of the Geul returns to the non-flooding situation. The tunnel is pumped after every use, which removes sludge as well as water.

To prevent water from refilling the tunnel from the outlet side in the non-flooding situation a solution had to be found. Another weir at the outlet side would reduce the discharge significantly, therefore the outlet will be designed the same as a side channel outlet; with a co-current channel. This channel will flow parallel to the Geul and gradually discharges water back into the Geul due to the difference in water level, see the hydraulic design of the outlet in Section 5.1.2.

Maintenance situation

During maintenance, temporary structures block the river's access to the tunnel. The temporary structures are barriers that can be slid into the specially designed spaces, see Figure 5.10. In this manner, the tunnel and the weir may be maintained. As the tunnel is emptied after each usage, sludge will not settle, making maintenance easier. Since the tunnel's area will not reduce, the maximum discharge via the tunnel will remain unchanged.

5.1.2. Hydraulic design

Using Systems Engineering, the hydraulic design is separated into various elements. First, the primary hydraulic design is calculated based on the discharges and water level, followed by the hydraulic design of the inlet, tunnel, and outlet.

Primary hydraulic design

The discharge through the flood bypass tunnel depends on the water level of the Geul at the inlet (open channel flow) and the difference in head between the water level in the shaft of the inlet and the outlet (pressurised flow). As the water must first flow over the weir before discharging through the tunnel, the water level of the Geul at its inlet is normative. Even though this is an iterative process, the hydraulic design of the open channel flow is initially explained, followed by the hydraulic design of the pressurised flow.

Open channel flow

Before the water flows into the tunnel, the water flows through the trash rack, followed by the flow over the Ogee weir. The hydraulic design that follows is explained in this section.

Trash rack
The trash rack reduces the energy level upstream of the crest of the Ogee weir as well as the difference in head. The head loss of the trash rack can be calculated using Equation 5.1 (Army Engineer Waterways Experiment Station, 1977).

$$\Delta H_{trashrack} = \frac{K_t U^2}{2g} \tag{5.1}$$

where:	$\Delta H_{trashrack}$	[m]	=	head loss through the trash rack
	K_t	[-]	=	head loss coefficient trash rack
	U	[m/s]	=	velocity of flow without trash rack
	g	[m/s^2]	=	gravitation constant

The value of the head loss coefficient of the trash rack K_t varies depending on the grid's size and shape, thus the design of the trash rack (Army Engineer Waterways Experiment Station, 1977). Appendix E.1 shows the value for certain forms and dimensions. The head loss coefficient of the trash rack is assumed to be 0.3, as it will likely exceed the most ideal shape but also encompasses a sizeable portion of the non-ideal shapes. In this case, the velocity through the tunnel is used as the velocity of flow without the trash rack which is iteratively calculated further in the section on pressurised flow. In reality, the velocity is lower due to the bigger area in this section and the open channel flow.

$$\Delta H_{trashrack} = \frac{0.3 \cdot (2.8)^2}{2 \cdot 9.81} = 0.12m$$

The head loss through the trash rack comes down to 0.12 m. The rule of thumb for a full trash rack is 0.1 m and emphasises this value.

Ogee weir
The calculation of the discharge over a weir is based on the general weir formula for free flow (Ankum, 1992), see Equation 5.2.

$$Q = c \cdot b \cdot H^{3/2} \tag{5.2}$$

where:	Q	[m^3/s]	=	discharge
	c	[-]	=	weir coefficient for free flow
	b	[m]	=	the width of the weir
	H	[m]	=	the (upstream) energy level above the crest level

The weir coefficient c depends on the shape of the crest. The broad-crested weir has a coefficient of c = 1.7, while the Ogee-crest has a coefficient c = 2.3. To increase the discharge a high coefficient is preferred, therefore the Ogee-crest shape is used, similar to the weir in San Antonio. The shape of this crest is seen in Figure 5.3.

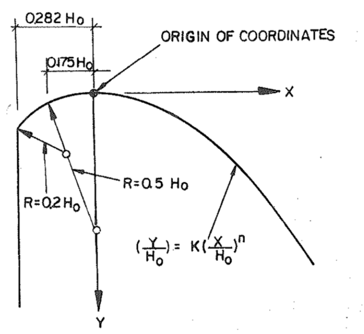


Figure 5.3: Ogee weir shape (Chen, 2015)

The (upstream) energy level above the crest H is determined using the colour code of Waterschap Limburg (2022b), see Figure 5.4. The water level will be in the non-flooding situation (green) up until

CD + 68.4 m. This situation is previously determined in Section 3.2.4. In this situation, the water is unwanted to flow into the tunnel. The bed level is located at CD + 66 m, therefore the crest height is 2.4 m to prevent water from flowing into the tunnel during a non-flooding situation.

During a flood, the water level will be equal to or larger than CD + 69.5 m. This correspondents to 1.1 m, see Figure 5.4. The head loss through the trash rack of 0.12 m reduces the energy level upstream, therefore the energy level upstream H is 0.98 m.

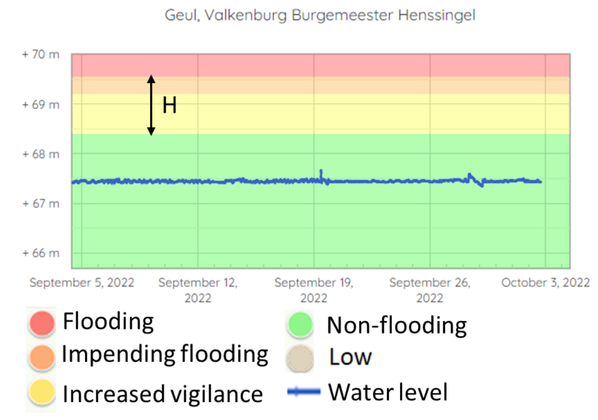
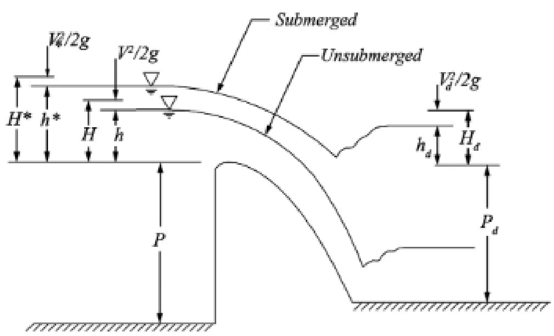


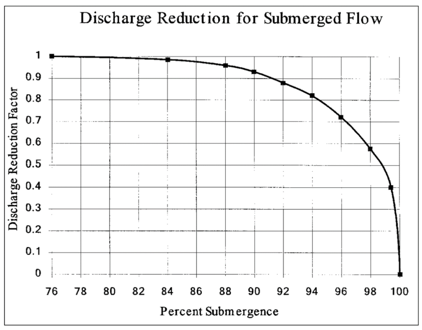
Figure 5.4: Energy level based on the water levels (Waterschap Limburg, 2022b)

The width of the weir, b, is varied to have enough of the difference in head left to create a free flow situation as the discharge significantly reduces if the weir is submerged, see Figure 5.5b. For a more detailed relation between free flow and submerged flow over different types of Ogee weirs, see Tullis (2011)'s paper.

To prevent submergence of the weir, the downstream water level should always be lower than $2/3H$ to satisfy the free flow condition, see Figure 5.5a (M. Z. Voorendt & Molenaar, 2019). This means that at least $1/3H$ which is 0.33 m has to be free to guarantee the free flow condition. For the difference in head of 2.65 m, subtracted by the head loss of the trash rack of 0.12 m, the difference in head available for the tunnel is 2.20 m. This is calculated in the pressurised flow section.



(a) Free flow and submerged Ogee weir (Tullis, 2011)



(b) Reduction of discharge depending on percentage of submergence (US Army Corps of Engineers, 2022)

Figure 5.5: Submerged weir

Cavitation conservatism
The design energy level above the crest H_D is set to 0.98 m, which is a conservative design choice according to Army Engineer Waterways Experiment Station (1977). The discharge without the risk of cavitation damage is possible until 130% of the energy level, see Equation 5.3. This conservative choice is made to make the system more robust than a once in 100 years flood as this water level was exceeded during the flood in 2021. This is elaborated in Chapter 7.

$$H = H_D \cdot 1.3 = 1.27m \tag{5.3}$$

where: H [m] = (upstream) energy level above the crest
H_D [m] = design energy level above the crest

Pressurised flow

The pressurised flow's velocity is calculated using the following formula, see Equation 5.4 Molenaar & Voorendt (2019). The available difference in head for the pressurised flow is 2.20 m to satisfy the free flow condition and the energy reduction due to the trash rack, see the open channel flow section.

$$\Delta H = \xi_{total}\frac{U^2}{2g} \Rightarrow U = \sqrt{\frac{2g\Delta H}{\xi_{total}}} \tag{5.4}$$

where: ΔH [m] = available difference in head (2.20 m)
ξ_{total} [-] = total pressure loss coefficient
U [m/s] = velocity
g [m/s^2] = gravitation constant

The total pressure loss coefficients are determined ξ separately and summed, see Equation 5.5 Molenaar & Voorendt (2019).

$$\xi_{total} = \sum \xi = \xi_{inflow} + \xi_{outflow} + \xi_f + \xi_{bends} \tag{5.5}$$

where: ξ_{total} [-] = total pressure loss coefficient
ξ_{inflow} [-] = inflow loss coefficient
$\xi_{outflow}$ [-] = outflow loss coefficient
ξ_f [-] = friction loss coefficient
ξ_{bends} [-] = bends loss coefficient

The pressure loss coefficients ξ are determined using coefficients from literature (Bengtson, 2016), see Appendix E.1.

1. Inflow loss coefficient ξ_{inflow}

The inflow shape loss coefficient and the bend loss coefficient are estimated to be 0.3 and 0.16 respectively, using coefficients from (Bengtson, 2016), see Appendix E.1.

$$\xi_{inflow} = \xi_{shapeinflow} + \xi_{bend} = 0.3 + 0.16 = 0.46$$

2. Outflow loss coefficient $\xi_{outflow}$

As the tunnel does not flow out in a lake the outflow parameter will be less than 1. The shape outflow parameter is therefore assumed to be 0.6. There is also another bend at the outlet which will have a coefficient of 0.16 using coefficients from Bengtson (2016), see Appendix E.1.

$$\xi_{outflow} = \xi_{shapeoutflow} + \xi_{bend} = 0.6 + 0.16 = 0.76$$

3. Friction loss coefficient ξ_f

The Colebrook formula is used to iterate the friction coefficient *f*, see Equation 5.6 Molenaar & Voorendt (2019).

$$f = \left\{-2\log_{10}\left[\frac{(k/D)}{3.7} + \frac{2.51}{\mathrm{Re}\left(f^{1/2}\right)}\right]\right\}^{-2} \tag{5.6}$$

where: f [-] = friction parameter
k [m] = roughness parameter
D [m] = diameter

Re [-] = Reynolds number

The roughness k is estimated to be 0.003 m to 0.0003 m as the tub will be smooth prefab concrete Bengtson (2016). Due to irregularities in the tunnel like seams, and segment transitions and as the smoothness will decrease over time a roughness parameter of 0.001 m is used.

The Reynolds number has to be estimated for this, see Equation 5.7 Molenaar & Voorendt (2019).

$$Re = \frac{DU\rho_{water}}{\mu} \tag{5.7}$$

where:
Re [-] = Reynolds number
D [m] = diameter
U [m/s] = velocity
ρ_{water} [kg/m^3] = density of water
μ [MPa · s] = dynamic viscosity of water

The velocity U is estimated using an iterative process of the discharge calculated by the weir formula as $Q = U \cdot A$, with A being the area of the tunnel. Density of water ρ_{water} and dynamic viscosity of water μ are respectively 1000 kg/m^3 and 0.001 $MPa \cdot s$.

This process is iterated to have the same assumed friction value,f, as the calculated one using the Colebrook formula. The ξ_f will then be calculated using the following formula, with a diameter D of 3.5 m and a length L of 875 m, see Equation 5.8 Molenaar & Voorendt (2019).

$$\xi_f = f\frac{L}{D} \tag{5.8}$$

where:
f [-] = friction parameter
L [m] = length of the tunnel
D [m] = diameter of the tunnel

This results in friction loss coefficient ξ_f of:

$$\xi_f = f\frac{L}{D} = 0.0149 \cdot \frac{900}{3.5} = 3.82$$

4. Bend loss coefficient ξ_{bend}
For the two bends in the tunnel; 90-degree bends are taken into account which each have a ξ_{bend} of R/D = 2 for 90 degrees. Using coefficients from Bengtson (2016), see Appendix E.1, 0.16 is used. Therefore with D = 3.5 m R is 7 m should be the radius of the bend.

$$\xi_{bends} = 2 \cdot \xi_{bend} = 2 \cdot 0.16 = 0.32$$

Total pressure loss coefficient ξ_{total}
The total pressure loss coefficient comes down to the following:

$$\xi_{total} = 0.46 + 0.76 + 3.82 + 0.32 = 5.36$$

Using Equation 5.4 the velocity through the tunnel can be calculated using the available difference in head for the pressurised flow and the total pressure loss coefficients. This results in a velocity of:

$$U = \sqrt{\frac{2g\Delta H}{\xi_{total}}} = \sqrt{\frac{2 \cdot 9.81 \cdot 2.20}{5.36}} = 2.8m/s$$

The discharge through the flood bypass tunnel can be calculated using Equation 5.9. The discharge comes down to 55 m^3/s.

$$Q = A \cdot U \tag{5.9}$$

where:	Q	[m^3/s]	=	discharge
	A	[m^2]	=	area of both tubes with a diameter of 3.5 m
	U	[m/s]	=	velocity

The discharge over the weir should be equal to the discharge through the tunnel which results in a width of 24.5 m, by rewriting Equation 5.2.

$$b = \frac{Q}{c \cdot H^{3/2}} = 24.5m$$

This results in a weir with a height of 2.4 m and a width of 24.5 m. The double tunnel has a discharge of approximately 55 m^3/s. Calculating the velocity back to the pressure loss coefficients gives the head loss in Table 5.1 and Figure 5.6.

Table 5.1: Difference in head for Design Alternative 1

	Difference in head [m]
$\Delta H_{trashrack}$	0.12
ΔH_{weir}	0.33
ΔH_{inflow}	0.19
$\Delta H_{outflow}$	0.31
$\Delta H_{friction}$	1.57
ΔH_{bends}	0.13
ΔH_{total}	2.65

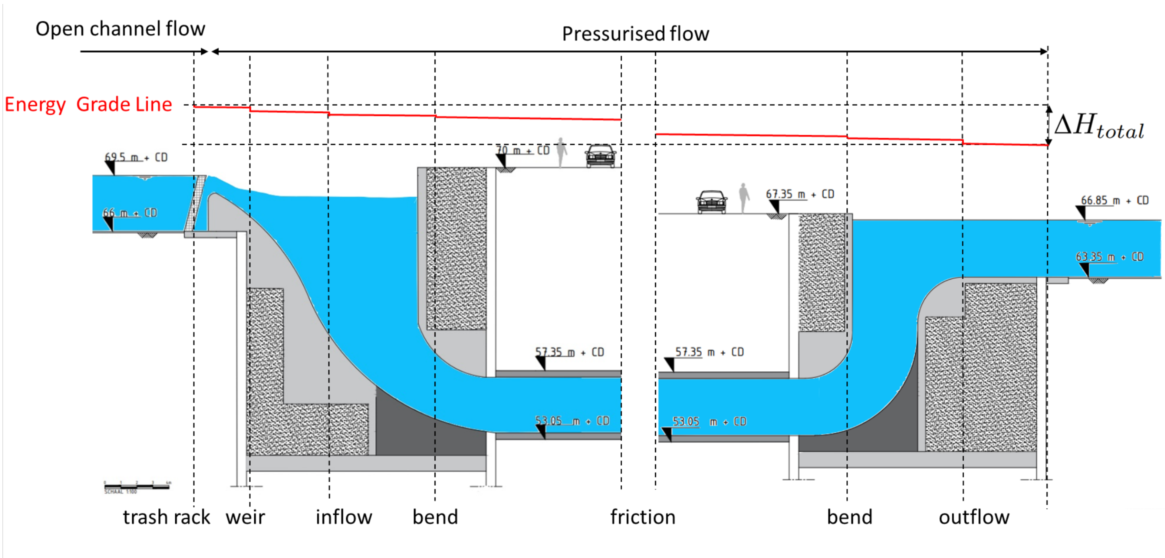


Figure 5.6: Head losses of Design Alternative 1

To verify this with the basis of design; for a 100-year return period, a discharge of 37 m^3/s is needed through the tunnel. With approximately 55 m^3/s this is quite over-designed. To limit this, two tubes with a diameter of 3 m were looked into. This comes down to 38 m^3/s with the same weir height due to the boundary conditions and a weir width of 17 m is needed. This will suffice the needed discharge of 37 m^3/s. This could be considered in further research, see Discussion Chapter 8. Alternatively, one tube with a 3.5 m diameter was proposed. This comes down to 27 m^3/s with the same weir height due to the boundary conditions and weir width of 12.3 m is needed. However, this will also not suffice, as it does not fulfil the needed discharge of 37 m^3/s. This is discussed further in the Discussion Chapter 8.

Inlet
In Figure 5.7 the location of the inlet is schematised. The inlet has a good connection to the Geul. The open water will have a railing to prevent people from falling in. Between the two tubes, there will be space for a pump installation. The trash rack will be in front of the weir and a small bridge will be connected to clean the trash rack. For the hydraulic design of the inlet, some calculations are done for the shape of the Ogee weir, the submersion of the tube and other design rules.

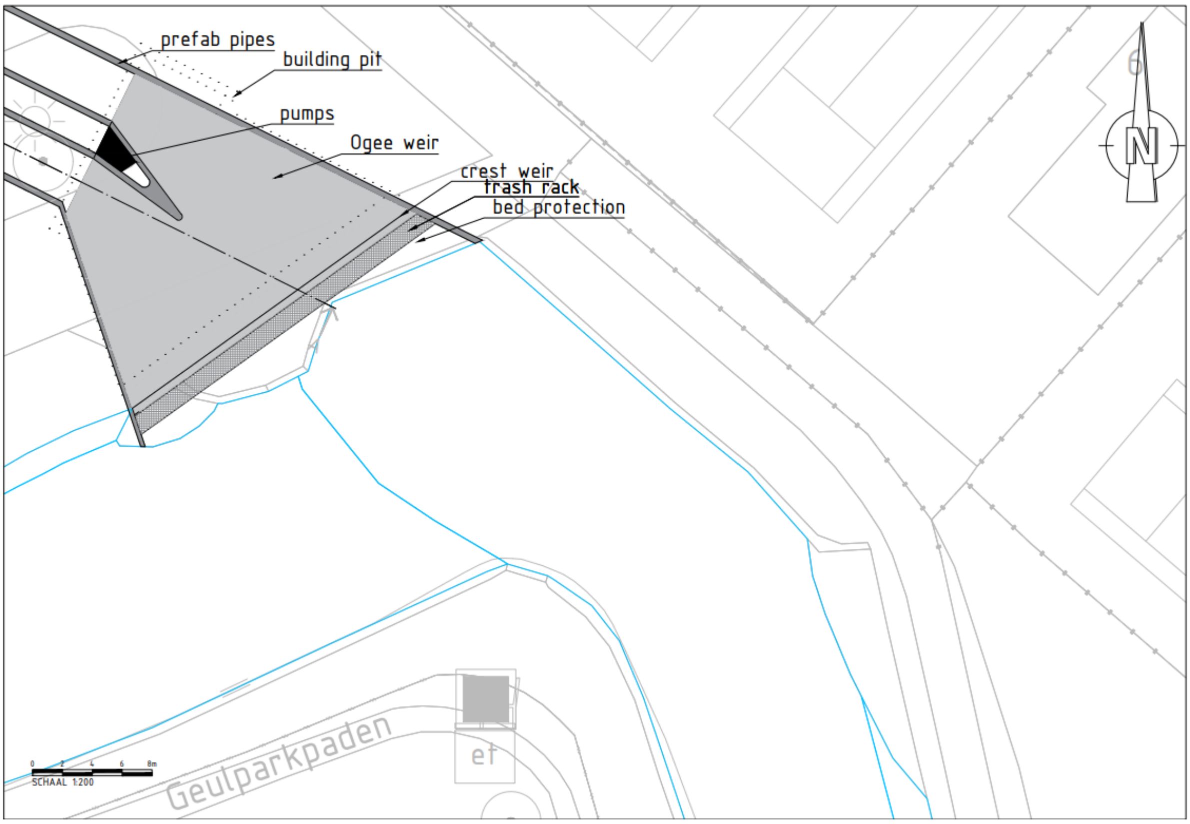


Figure 5.7: Topview inlet

Shape of the Ogee weir
As the main advantage of the Ogee weir is its increased discharge capacity, the shape of the Ogee weir is crucial for its performance. The design formulae of US Army Corps of Engineers (2022) determine the shape's design.

The design head, H 0.98 m, and approach depth, P 2.4 m, determine the shape. The Ogee weir is divided into four different parts: The first part is determined by Equation 5.10.

$$Y = -\sqrt{\left(1 - \frac{X^2}{A_{Ogee}^2}\right) B^2} + B \tag{5.10}$$

where:	Y	[m]	=	Y-coordinate
	X	[m]	=	X-coordinate
	A_{Ogee}	[-]	=	constant depending on ratio H and P (in this case 0.308)
	B	[-]	=	constant depending on ratio H and P (in this case 0.182)

The second part is determined by Equation 5.11.

$$Y = \frac{X^{1.85}}{K \cdot H^{0.85}} \tag{5.11}$$

where: Y [m] = Y-coordinate
X [m] = X-coordinate
K [-] = constant (in this case 2)
H [m] = the (upstream) energy level above the crest level

The third part is a constant slope. This slope is varied over the width of the weir to suit the space available from the crest of the weir to the start of the tunnel tube, see Figure 5.7. The bed slope will be less steep than 2;1.

The fourth part is a circle with a radius which is a quarter of the height. The height is 15.35 m, therefore the circle has a radius of 3.8 m.

Submersion of the tube
Minimum submergence is required to prevent strong air core vortexes from entering the tunnel. The equations are empirical to determine the critique submersion (American National Standard Institute, 1998). The submersion formula is based on the Froude number:

$$F_D = \frac{V}{\sqrt{gD}} \tag{5.12}$$

where: F_D [-] = Froude number
V [m/s] = Velocity
D [m] = Diameter
g [m/s^2] = gravitational acceleration

The minimum submergence, S, is calculated using (Hecker, G.E., 1987):

$$S \geq D\,(1 + 2.3F_D) \tag{5.13}$$

where: S [m] = Submergence
F_D [-] = Froude number
D [m] = Diameter

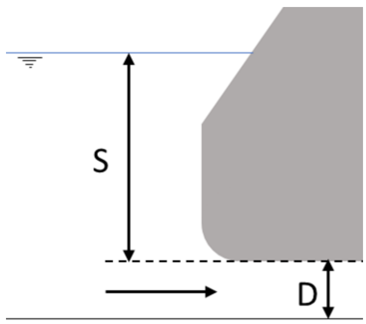


Figure 5.8: Submersion; based on (Schleiss, 2005)

The minimum submergence is calculated, using Equation 5.12 and Equation 5.13.

$$F_D = \frac{V}{\sqrt{gD}} = \frac{V}{\sqrt{gD}} = 0.49$$

$$S \geq D\,(1 + 2.3F_D) = D\,(1 + 2.3F_D) = 7.4m$$

As the depth from the design water level to the top of the tube is 12.6 m, and the calculated minimum submergence is 7.4 m, strong air core vortexes are prevented from entering the tunnel when the tunnel is filled.

Other design rules

Some other design rules are elaborated here. The transition length from a square to a round tube is calculated using Equation 5.14 (Schleiss, 2005):

$$L_{transition} = \frac{D}{2} \tag{5.14}$$

where: $L_{transition}$ [m] = transition length from a square to round tube
D [m] = Diameter

The length of the building pit will be the transition from square to round tube, this would suffice with 1.75 m.

$$L_{transition} = \frac{D}{2} = \frac{3.5}{2} = 1.75m$$

The minimal width of the inlet at the entrance is calculated using Equation 5.15 (Schleiss, 2005):

$$W_{entrance} = 1.65 \cdot D \tag{5.15}$$

where: $W_{entrance}$ [m] = minimal width of the inlet at the entrance is
D [m] = Diameter

The minimal width of the inlet at the entrance will be wider than 5.78 m due to the weir.

$$W_{entrance} = 1.65 \cdot D = 1.65 \cdot 3.5 = 5.78m$$

Tunnel

Horizontal and vertical tubes promote air transport, therefore the tubes are designed vertically and horizontally. The tunnel is located at least 10 m below the surface level, and due to the design choice of a horizontal tube, the outlet tunnel depth is normative. The following paragraph provides background information on air transport in tubes and supports this design choice.

Air transport in a tube

A descending tube transports air less efficiently, which means the air bubble can lead to significant head losses. The stagnation of air bubbles occurs when the tube lays steeper than the hydraulic gradient. Due to this phenomenon, the ideal profile of the tube without a chance of air entrapment would be a flat or ascending profile as stagnation only occurs in descending pipe sections. If a descending pipeline is necessary, steep angles of inclination are preferred (> 60°, with 90° slopes achieving the greatest gas transport). Research shows that gas can be discharged more quickly at steep angles of inclination.

From research by Lubbers (2007), the required flow number for gas volume transport in a vertical pipeline (angle of inclination is 90°) is 0.4 and is therefore significantly smaller than the required flow number at 60°, see Figure 5.9. Thus, gas bubbles are most efficiently descended in a vertical pipe. For an elaborate explanation of this phenomenon see Appendix E.2.

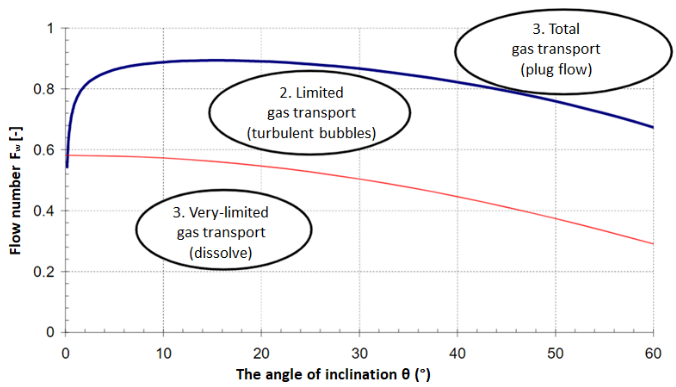


Figure 5.9: Air transport in a tube; translated (Tukker et al., 2012)

For the estimated flow number of 0.49. the angle of the decreasing slope of the tube should be approximately bigger than 40 degrees to achieve a limited amount of transport of air, see Figure 5.9. However, to promote total gas transport a bigger slope is needed. This is achieved for an angle of 90 degrees (vertical tube) if the flow number is 0.4 or higher, which is the case. To exclude air entrapment between the inlet and the outlet, thus head loss, the ideal profile of the tube would be a flat or ascending profile. To reduce costs (boring deeper is always more expensive) the tube is chosen to be horizontal between the inlet and outlet.

Outlet
In Figure 5.10 the location of the outlet is schematised. The structure has a railing to prevent people from falling in and there is space for temporary structures if maintenance is required. The outlet has a good connection to the Geul with a co-current channel to prevent water streaming from the outlet into the tunnel during a non-flooding situation so the tunnel can stay empty. The co-current channel also increases the discharge downstream and prevents water from flowing back into the city. The co-current channel will be constructed up until the determined length where the water levels meet. This is based on a basic equation using the estimated slope of the river profile, see Equation 5.16 and 5.17. The co-current channel is illustrated in Figure 5.11.

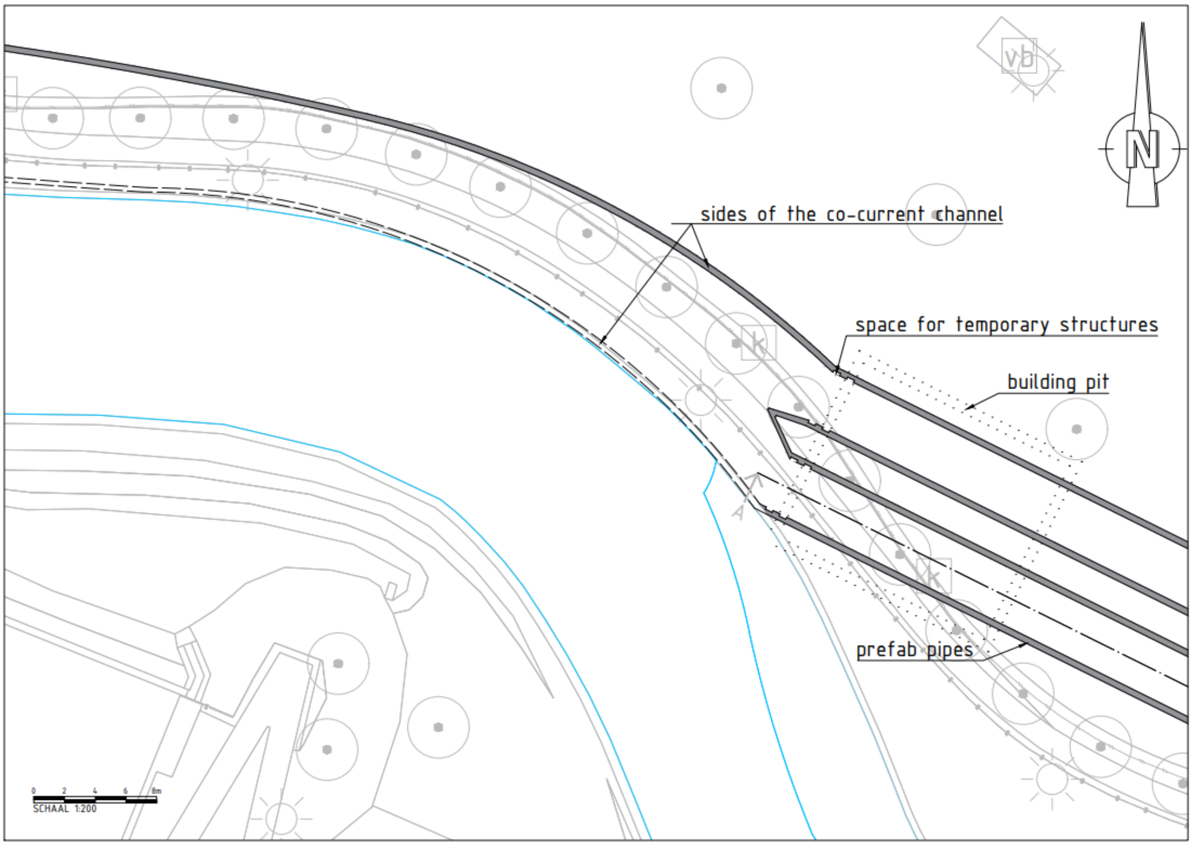


Figure 5.10: Topview outlet

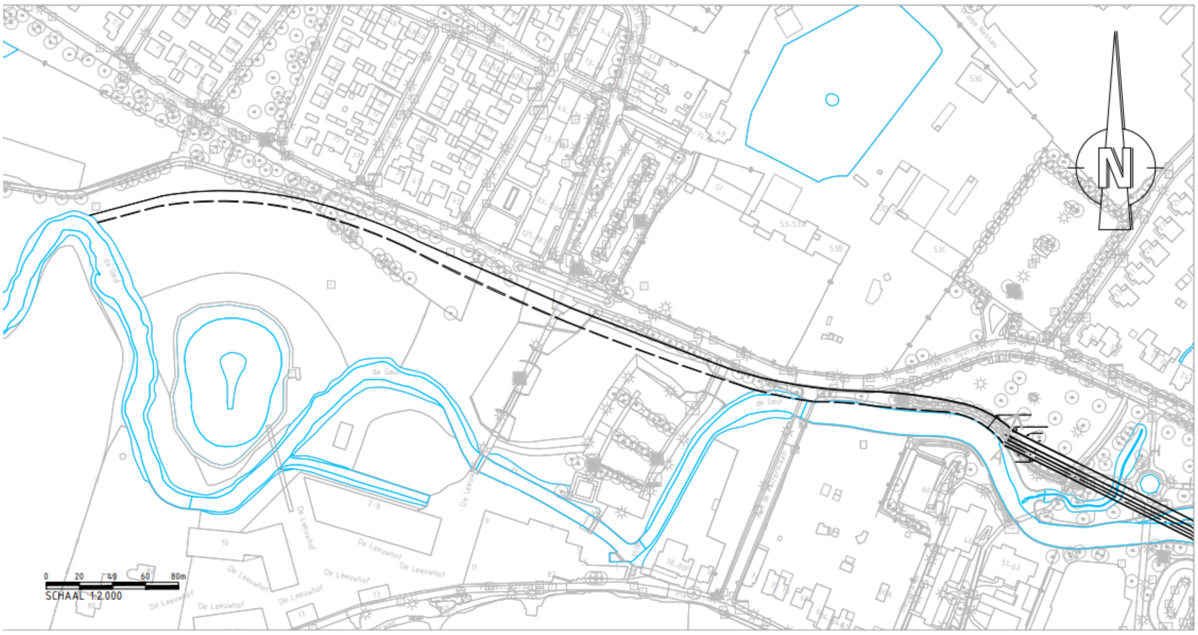


Figure 5.11: Topview co-channel

To get an estimate of the length of the co-current channel the length following the Geul is 900 m in between the location of the inlet and outlet and the slope is assumed to be linear to simplify the equation. This is calculated as follows:

$$i_{GeulatValkenburg} = \frac{\Delta b}{l_{in-tooutlet}} \tag{5.16}$$

where:

$i_{GeulatValkenburg}$	[-]	=	gradient of the bed slope of the Geul
Δb	[m]	=	difference in bed level elevation
$l_{in-tooutlet}$	[m]	=	length of the river between the in- and outlet location

$$i_{GeulatValkenburg} = \frac{\Delta b}{l_{Geul}} = \frac{CD + 66m - CD + 63.35m}{900m} = \frac{2.65}{900}$$

$$l_{co-current} = \frac{\Delta h}{i_{GeulatValkenburg}} \tag{5.17}$$

where:

$l_{co-current}$	[m]	=	length of the co-current channel
Δh	[m]	=	difference in water level of location outlet until bedlevel

$$l_{co-current} = \frac{\Delta h}{i_{GeulatValkenburg}} = \frac{2.65}{\frac{2.65}{900}} \approx 900m$$

The co-current channel will follow the current side channel downstream and end further downstream where the side channel is reconnected to the Geul. The length of this part of the Geul is 900m long. This means the co-current channel runs parallel to the Geul.

5.1.3. Final Design Alternative 1

Following a brief explanation of the construction process, the final design is presented and summarised. The design is shown as an attachment to the thesis.

Construction process

The initial phase of construction is for both design variants the same. First, the diaphragm walls are constructed, followed by the excavation of the building pits on both the inlet and outlet sides. The building pits are approximately 15 meters long, 10 meters wide, and 10 meters deep. In both construction pits, an underwater concrete floor is constructed. After pumping out the groundwater from the pit, a thrust wall is created to provide a solid surface from which to jack the pipes. Following the lowering of the tunnel machine and pipes into the drive shaft and onto a jacking rig, an entry sealing is constructed in the tunnel through which the tunnel machine and pipes can pass. The tunnel machine is initially pushed forward by powerful hydraulic jacks tunnelling through the ground to the other shaft. When the machine reaches the length of an individual tunnel tube, excavation is stopped and the first jacking pipe is pushed up to the back of the tunnel machine, where it is connected to the machine and the process continues. The process continues until the tunnel machine is jacked into the other shaft to complete the excavation. The equipment and machinery are removed, and the shafts are completed as inlet and outlet. The inlet and outlet are created on-site in phases to minimise the use of concrete and, whenever possible, to use sand, see Figure 5.12 and 5.12. There is space for a temporary pump in the shaft of the inlet, between the tunnel entrances. After the inlet and outlet in the shafts have been created, the co-current downstream of the outlet and bed protection is constructed. To do this, two bridges must be replaced. After this, the trash rack and hash marks are constructed and the tunnel can operate.

Summary

In summary, the first design alternative is based on the passive flood bypass tunnel. The passive inlet will consist of an Ogee weir inlet and an angled trash rack. The water level will be controlled by the 2.4-meter-tall and 24.5-meter-wide weir. When in function, the water will flow into the tunnel and inevitability cause air to enter the tunnel as well. The air is completely transported with the flow due to the horizontal and vertical positions of the tubes to avoid pressure peaks and the reduction of the discharge capacity of the flood bypass tunnel. Once the tunnel is filled, the flow is pressurised. After a flood, the remaining water will be pumped out. A co-current channel is designed to prevent water from refilling the tunnel from the outlet side in the non-flooding situation. This channel will flow parallel to the Geul and gradually discharges water back into the Geul due to the difference in water level. The flood bypass tunnel has a discharge capacity of approximately 55 m^3/s. The overview of the location of the flood bypass tunnel is illustrated in Figure 5.14.

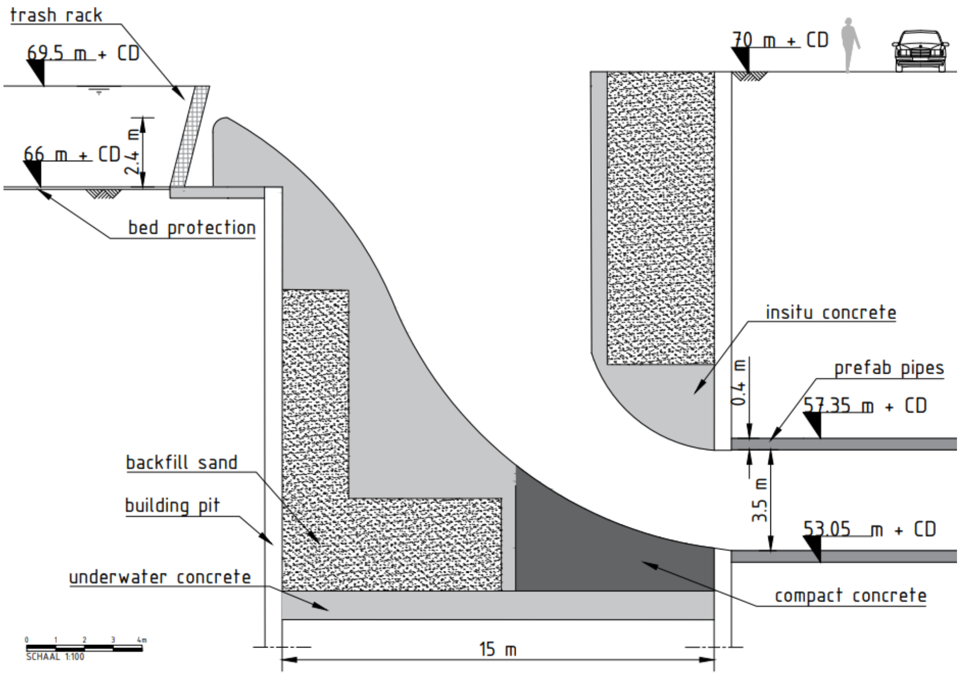


Figure 5.12: Detailed cross-section of the inlet

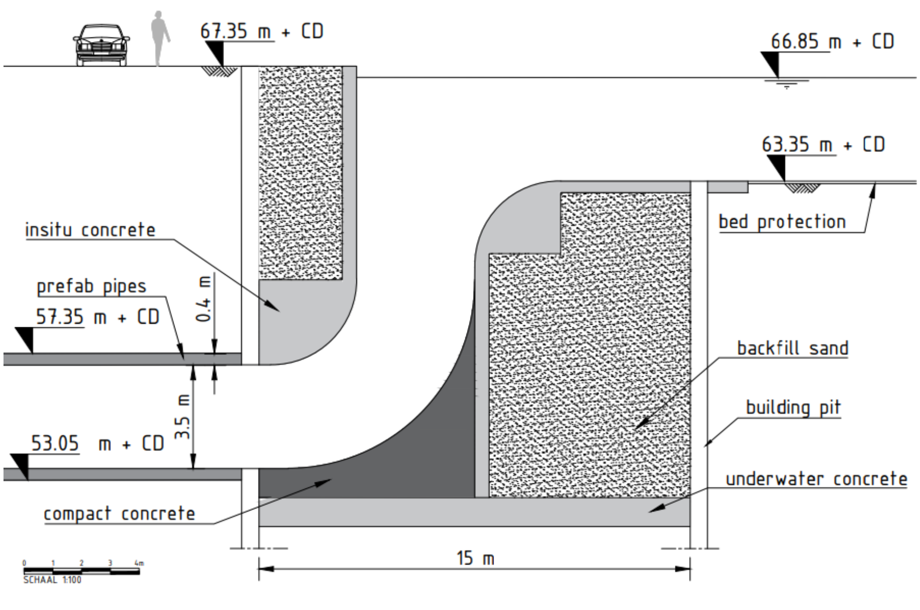


Figure 5.13: Detailed cross-section of the outlet

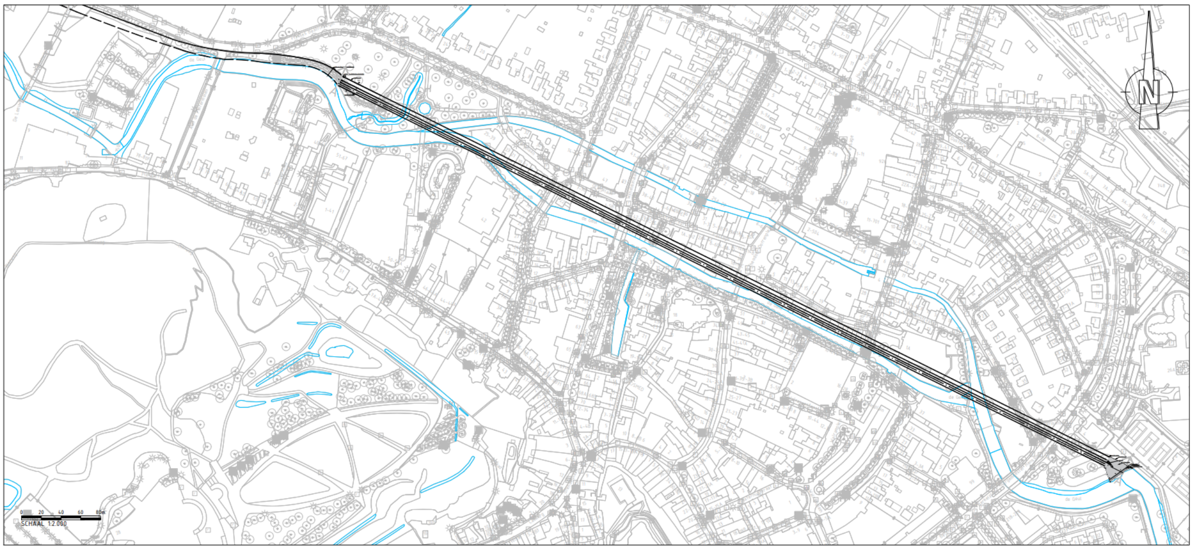


Figure 5.14: Top view of the total design

5.2. Design Alternative 2: active flood bypass tunnel

The design alternative is developed iteratively to find a working combination of the operation and maintenance situation while optimizing head losses and preventing air from entering the tunnel to avoid pressure peaks. This design alternative is inspired by the Sarneraa and Thun tunnel using a flat gated in- and outlet with a trash rack at the inlet, see Figure 5.15. Just as in the reference project the tunnel will be filled and closed off when not in function. The design alternative is based on an active inlet and thus requires a moving gate to control the flow. There is also a gate at the outlet to prevent water from flowing back in. Because the gate has mechanical parts, it is less reliable, but it is easier to control.

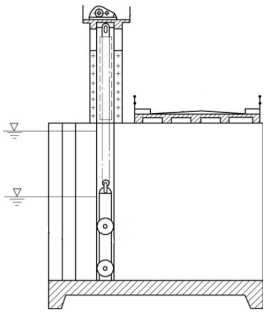


Figure 5.15: Vertical moving flat gate (Chen, 2015)

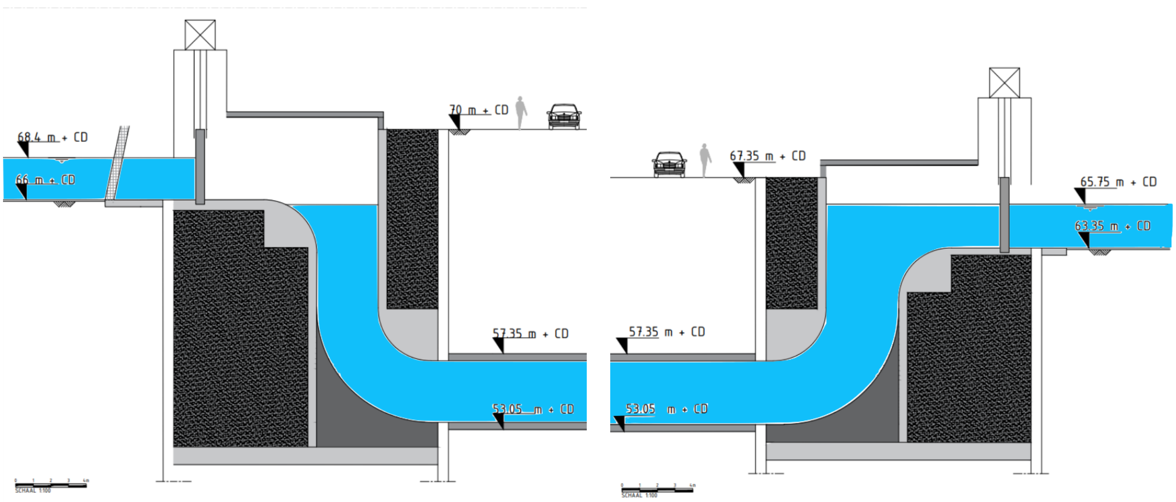
The operation of the design alternative is first explained in Section 5.2.1, after which the design alternative is hydraulically designed in Section 5.2.2 which leads to the final design alternative in Section 5.2.3.

5.2.1. Operation and maintenance of the tunnel

In this section, the operation and maintenance of the tunnel are described in different situations: non-flooding, flood, after the flood, and maintenance.

Non-flooding situation

In the non-flooding situation, the water is contained in the tunnel, but the gates of the inlet and the outlet are closed, see Figure 5.16. The water will only flow through the Geul until the water level of Chart Datum (CD) + 68.40 m at the inlet is reached, or until a flood is predicted using an early automatic detection system. These water levels are determined using the colour code of Waterschap Limburg (2022b), as described in Section 3.2.4.



(a) Inlet in non-flooding situation

(b) Outlet in non-flooding situation

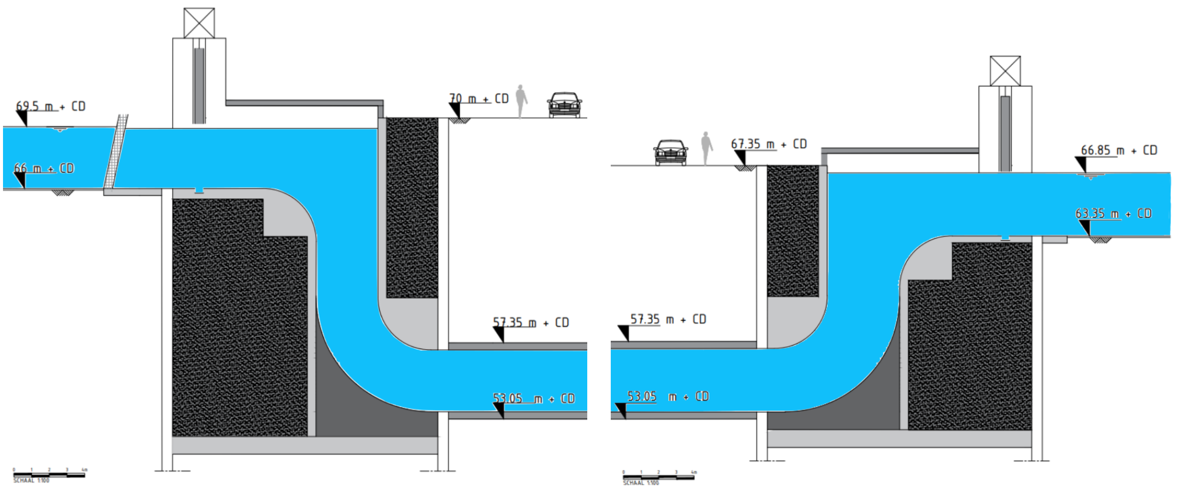
Figure 5.16: Design alternative 2 in the non-flooding situations

Flood situation

When the water level of the Geul at the inlet exceeds the non-flooding water level of CD + 68.40 m, or when a flood is predicted using an early automatic detection system, the gates at the inlet and outlet side are opened and the water will flow through the angled trash rack and into the tunnel. The trash rack prevents debris from entering the tunnel and averts blockages as debris floats to the top of the trash rack.

As the tunnel is already filled with water and the tube is submerged, air will not enter the tunnel, this is hydraulically designed in Section 5.2.2 for the inlet. The tunnel functions as a pressurised pipe, therefore no pressure peaks will happen, see Figure 5.17. The difference in head between the water level at the inlet and the outlet discharges the water through the tunnel.

If however in any circumstance air enters the tunnel, for example refilling the tunnel after maintenance, the horizontal position of the tunnel will remove this air as it promotes total transport of the air bubbles with the flow for the calculated flow number, see the hydraulic design of the tunnel in Section 5.2.2.



(a) Inlet in flood situation

(b) Outlet in flood situation

Figure 5.17: Design alternative 2 in the flood situations

After flood situation

After a flood, once the water level of the Geul returns to the non-flooding situation, the gates at the in- and outlet will close. The water will stay inside the tunnel to keep the flood bypass tunnel pressurised. The tunnel is closed off from its environment, to prevent stench from the still-standing water, and ensure no breeding of mosquitoes. The flood bypass tunnel is flushed twice a year, see the Section on the maintenance situation. The water could be used during drought, but this is not looked into any further, see Discussion Chapter 8.

Maintenance situation

The gates are tested twice a year (spring and autumn) and after use, they need to be checked for damages as well (Waterschap Limburg, personal communications, December 6, 2022). With the testing of the gates, the system is flushed, water is refreshed and will take some sludge with it in the process. However, settled sludge might not flush out entirely, which decreases the diameter of the tunnel and decreases the discharge. Once every couple of years the tunnel, therefore, is pumped out. A temporary pump can be installed at the inlet between the two tubes, this will be the lowest point of the tunnel. The tunnel will be blocked from the river's access with temporary structures, so the tunnel and also the gates can be maintained. The temporary structures are barriers that can be slid into the specially designed spaces, see Figure 5.19 and 5.20. The tunnel is emptied and sludge is removed.

After maintenance, however, the tunnel needs to be re-filled to make sure the flood bypass tunnel is pressurised for when a flood is coming. This is done in a controlled matter using the gates. It works similarly to the filling stage of the tunnel of the first design alternative, described in the previous section.

The horizontal position of the tunnel removes any air that might enter the tunnel, as it promotes total transport of the air bubbles with the flow for the calculated flow number, see the hydraulic design of the tunnel in Section 5.1.2.

5.2.2. Hydraulic design

Using Systems Engineering, the hydraulic design is separated into various elements. First, the primary hydraulic design is calculated based on the discharges and water level, followed by the hydraulic design of the inlet, tunnel, and outlet.

Primary hydraulic design

The discharge through the flood bypass tunnel depends only on the difference in head between the water level at the inlet and the outlet. Therefore the hydraulic design depends fully on the difference in head through the tunnel.

Open channel flow

Before the water flows into the tunnel, the water flows through the trash rack. The hydraulic design that follows is explained in this section.

Trash rack

The trash rack reduces the difference in head. The head loss of the trash rack can be calculated using Equation 5.1 (Army Engineer Waterways Experiment Station, 1977). Here the head loss coefficient of the trash rack is assumed to be 0.3. The velocity through the tunnel is used as the velocity of flow without the trash rack which is iteratively calculated further in the section on pressurised flow.

$$\Delta H_{trashrack} = \frac{0.3 \cdot (3.0)^2}{2 \cdot 9.81} = 0.14m$$

The head loss through the trash rack comes down to 0.14 m. The rule of thumb for a full trash rack is 0.1 m and emphasises this value. For the difference in head of 2.65 m, subtracted by the head loss of the trash rack of 0.14 m, the difference in head available for the tunnel is 2.51 m. This is calculated in the pressurised flow section.

Pressurised flow

The pressurised flow's velocity is calculated using the following formula, see Equation 5.18 Molenaar & Voorendt (2019). The available difference in head for the pressurised flow is 2.51 m, see the open channel flow section.

$$\Delta H = \xi_{total} \frac{U^2}{2g} \Rightarrow U = \sqrt{\frac{2g\Delta H}{\xi_{total}}} \tag{5.18}$$

where:	ΔH	[m]	=	available difference in head (2.51 m)
	ξ_{total}	[-]	=	total pressure loss coefficient
	U	[m/s]	=	velocity
	g	[m/s^2]	=	gravitation constant

The pressure loss coefficients ξ are determined separately using coefficients from the literature (Bengtson, 2016) and summed, see Equation 5.5.

1. Inflow loss coefficient ξ_{inflow}

The inflow shape loss coefficient and the bend loss coefficient are estimated to be 0.4 and 0.16 respectively, using coefficients from Bengtson (2016), see Appendix E.1.

$$\xi_{inflow} = \xi_{shapeinflow} + \xi_{bend} + \xi_{trashrack} = 0.4 + 0.16 + = 0.56$$

2. Outflow loss coefficient $\xi_{outflow}$

As the outflow does not flow out in a lake the parameter will be less than 1 for now the outflow will be estimated to be 0.6, see Figure 5.20. There is also another bend which will have a coefficient of 0.16 m.

$$\xi_{outflow} = \xi_{outflow} + \xi_{bend} = 0.6 + 0.16 = 0.76$$

3. Friction loss coefficient ξ_f
The Colebrook formula is used to iterate the friction coefficient *f*, see Equation 5.6. The roughness k is estimated to be 0.003 m to 0.0003 m as the tub will be smooth prefab concrete Bengtson (2016). As the tunnel will be less smooth over time a roughness parameter of 0.001 m is used.

The Reynolds number will be estimated using the following formula. The velocity U is estimated using an iterative process of the discharge calculated by the weir formula as $Q = U \cdot A$, with A being the area of the tunnel. Density of water ρ_{water} and dynamic viscosity μ are respectively 1000 kg/m^3 and μ 0.001 $MPa \cdot s$, see Equation 5.7.

This process is iterated to have the same assumed f value as the calculated one using the Colebrook formula. The ξ_f will then be calculated using the following formula, with a diameter D of 3.5 m and a length L of 875 m, see equation 5.8.

This results in friction loss coefficient ξ_f of:

$$\xi_f = f\frac{L}{D} = 0.0149 \cdot \frac{900}{3.5} = 3.82$$

4. Bend loss coefficient ξ_{bend}
For the two bends in the tunnel; 90-degree bends are taken into account which each have a ξ_{bend} of R/D = 2 for 90 degrees. Using coefficients from Bengtson (2016), see Appendix E.1, 0.16 is used. Therefore with D = 3.5 m R is 7 m should be the radius of the bend.

$$\xi_{bends} = 2 \cdot \xi_{bend} = 2 \cdot 0.16 = 0.32 \tag{5.19}$$

Total pressure loss coefficients ξ_{total}
The total pressure loss coefficient comes down to the following:

$$\xi_{total} = 0.56 + 0.76 + 3.82 + 0.32 = 5.46$$

Using Equation 5.18 the velocity through the tunnel can be calculated using the available difference in head for the pressurised flow and the total pressure loss coefficients. This results in a velocity of:

$$U = \sqrt{\frac{2g\Delta H}{\xi_{total}}} = \sqrt{\frac{2 \cdot 9.81 \cdot 2.51}{5.46}} = 3.0m/s$$

The discharge through the flood bypass tunnel can be calculated using Equation 5.9. This results in a double tunnel which has a discharge of approximately 58 m^3/s. Calculating the velocity back to the pressure loss coefficients gives the head loss in Table 5.2 and Figure 5.18.

Table 5.2: Difference in head for Design Alternative 2

	Difference in head [m]
$\Delta H_{trashrack}$	0.14
ΔH_{inflow}	0.26
$\Delta H_{outflow}$	0.35
$\Delta H_{friction}$	1.76
ΔH_{bends}	0.15
ΔH_{total}	2.65

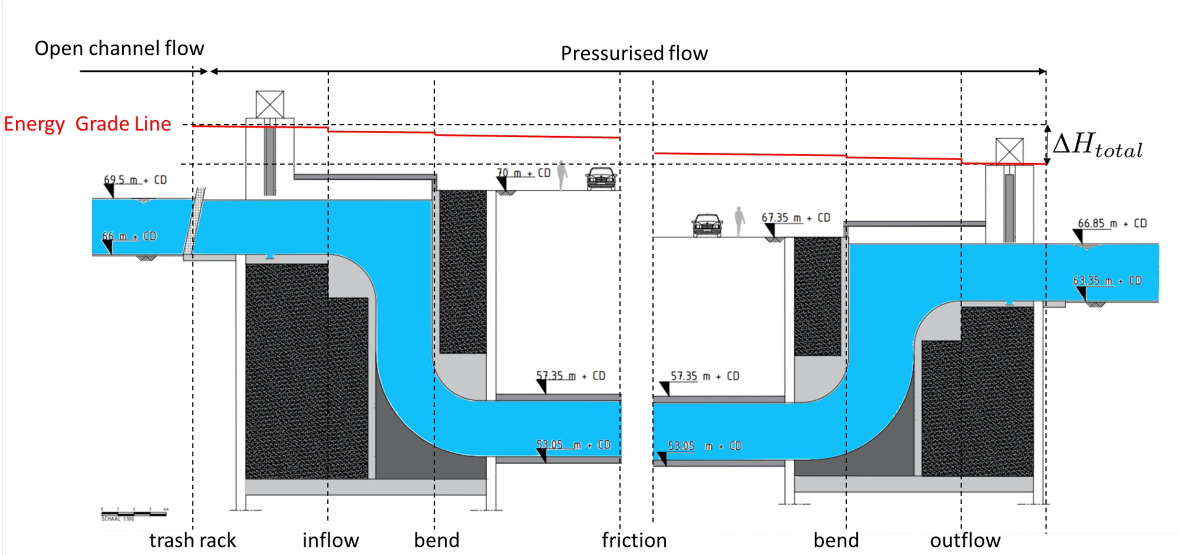


Figure 5.18: Head losses of Design Alternative 2

To verify this with the basis of design; for a 100-year return period, a discharge of 37 m^3/s is needed through the tunnel. With approximately 58 m^3/s this is quite over-designed. To limit this, there has been looked at two tubes of a diameter of 3 m which comes down to 40 m^3/s. This will suffice the needed discharge of 37 m^3/s. This could be considered in further research, see Discussion Chapter 8. There has also been looked at one tube of 3.5 m which comes down to 29 m^3/s. This will not suffice the needed discharge of 37 m^3/s. This is discussed further in the Discussion Chapter 8.

Inlet

In Figure 5.19 the location of the inlet is schematised. The inlet has a good connection to the Geul. The inlet will be completely covered to prevent stench from still-standing water and mosquito breeding. Between the two tubes, there will be space for a pump installation. The trash rack will be in front of the gates and a small bridge will be connected to clean the trash rack. There is also space to place a temporary slide gate for maintenance. For the hydraulic design of the inlet the submersion of the tube and other design rules are calculated.

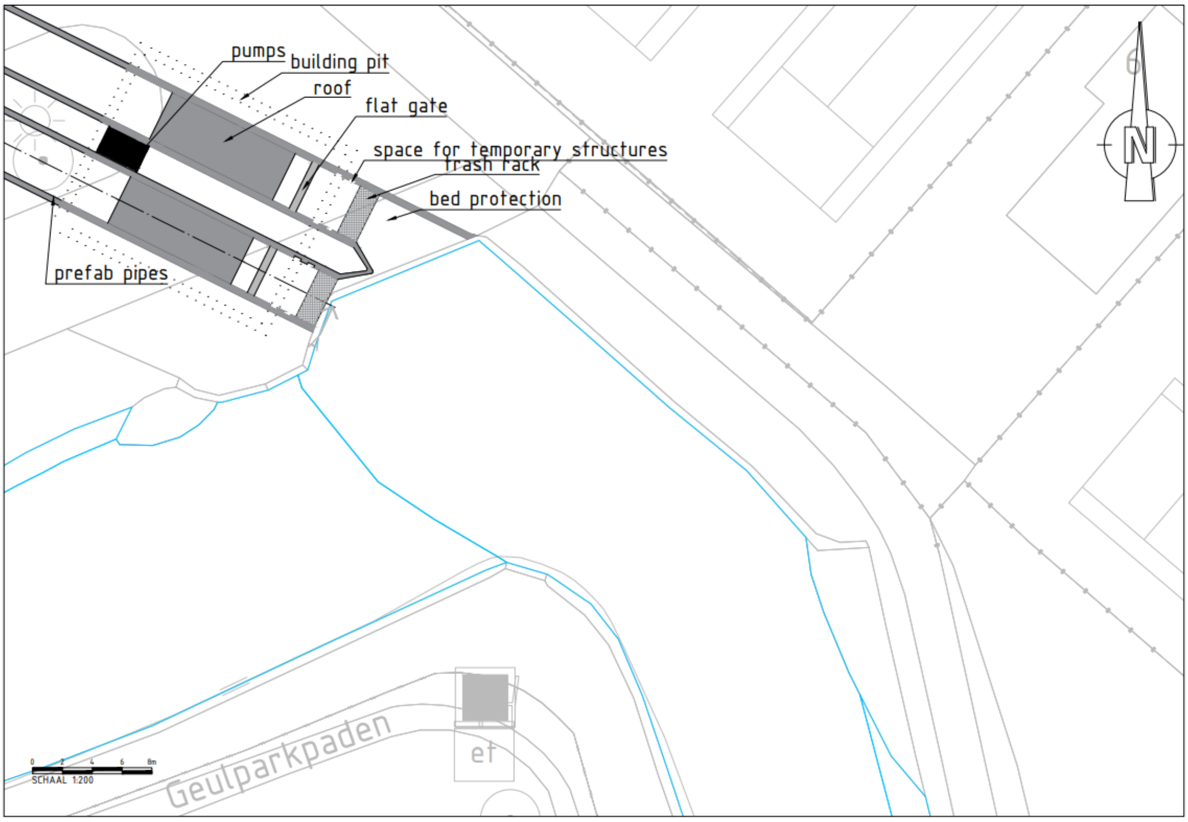


Figure 5.19: Topview inlet

Submersion

The minimum submergence is calculated, using Equation 5.12 and Equation 5.13.

$$F_D = \frac{V}{\sqrt{gD}} = \frac{V}{\sqrt{gD}} = 0.51$$

$$S \geq D\left(1 + 2.3F_D\right) = D\left(1 + 2.3F_D\right) = 7.6m$$

As the depth from the design water level to the top of the tube is 12.6 m, and the calculated minimum submergence is 7.6 m, strong air core vortexes are prevented from entering the tunnel.

Other design rules

The other design rules apply here as well. The transition from square to round tube will be located just after the flat gate, this would suffice with 1.75 m, see Equation 5.14.

$$L_{transition} = \frac{D}{2} = \frac{3.5}{2} = 1.75m$$

The minimal width of the inlet at the entrance should also be wider than 5.78 m per pipe, see Equation 5.15. This would suffice, see Figure 5.19.

$$W_{entrance} = 1.65 \cdot D = 1.65 \cdot 3.5 = 5.78m$$

Tunnel

The tunnel is full to keep the flood bypass tunnel pressurised in the non-flooding situation. The tunnel still has to be filled for example after maintenance. To promote air transport during the filling of the tunnel the horizontal and vertical tubes are here chosen as well. Here, also the outlet tunnel depth is normative. The following paragraph supports this design choice.

Air transport in a tube
For the estimated flow number of 0.51. the angle of the decreasing slope of the tube should be approximately bigger than 35 degrees to achieve a limited amount of transport of air, see Figure 5.9. However, to promote total gas transport a bigger slope is needed. This is achieved for an angle of 90 degrees (vertical tube) if the flow number is 0.4 or higher, which is the case. To exclude air entrapment between the inlet and the outlet, thus head loss, the ideal profile of the tube would be a flat or ascending profile. To reduce costs (boring deeper is always more expensive) the tube is chosen to be horizontal between the inlet and outlet.

Outlet
In Figure 5.20 the location of the outlet is schematized. The outlet will be completely covered to prevent stench from still-standing water and mosquito breeding and there is space for temporary structures if maintenance is required. The outlet has a good connection to the Geul and the width of the Geul is widened here to increase its capacity and prevent water from streaming back into the city during a flood. The widening is executed up until the side channel downstream of Valkenburg, and the capacity of the side channel is increased as well.

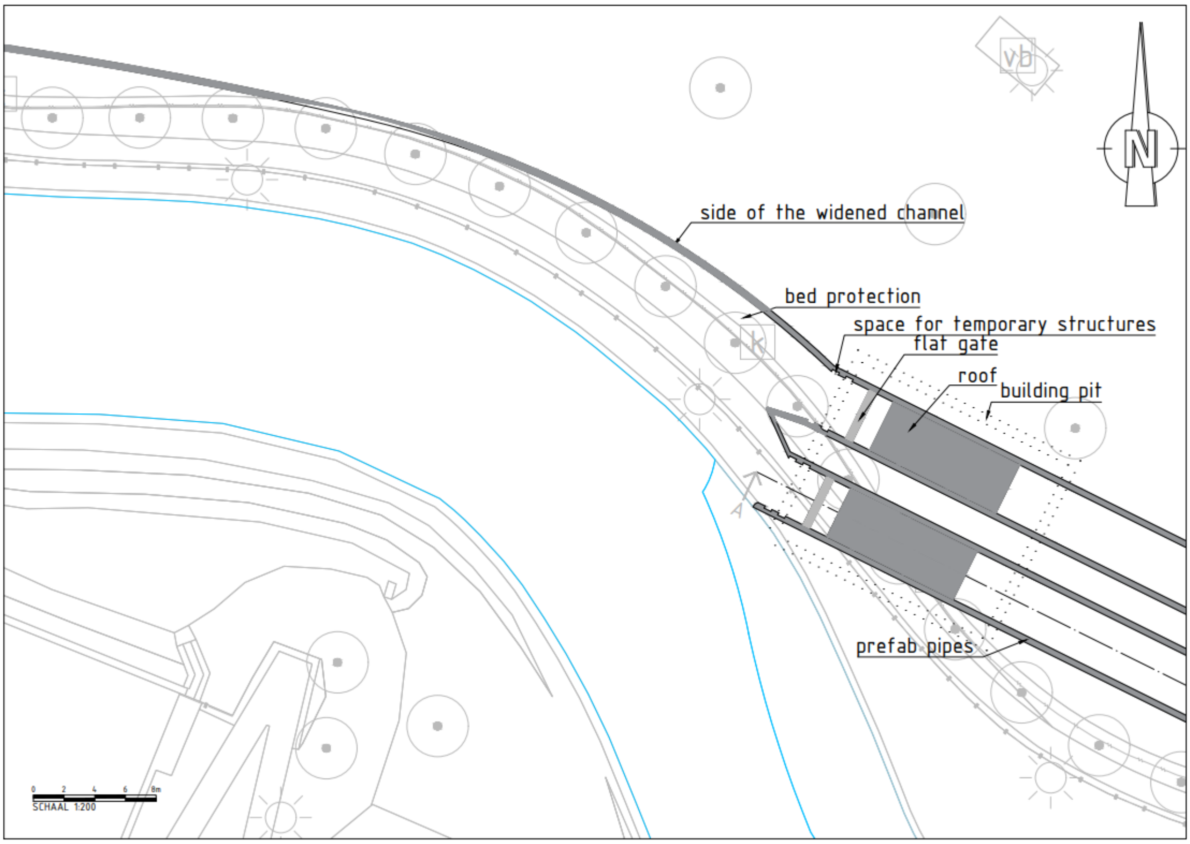


Figure 5.20: Topview outlet

5.2.3. Final Design Alternative 2
Following a brief explanation of the construction process, the final design is presented and summarised. The design is shown as an attachment to the thesis.

Construction process
The initial phase of construction is for both design variants identical as this involves the pipe jacking technique. See Section 5.2.3 for further explanation of this process. As the construction process is completed, the equipment and machinery are removed. The inlet and outlet are also created on-site in phases to minimise the use of concrete and, whenever possible, to use sand, see Figure 5.21 and 5.21. There is space for a temporary pump in the shaft of the inlet, between the tunnel entrances.

After creating the inlet and outlet in the shafts, the control houses, which include MEP (mechanical, electrical, and plumbing), are constructed and the flat gates are installed. In addition, Geul is widened downstream of the outlet in order to increase its discharge capacity, and bed protection is installed. To do this, two bridges must be rebuilt. After this the trash rack and hash marks are constructed and the tunnel can operate.

Summary

In summary, the second design alternative is based on an active flood bypass tunnel. The active inlet will consist of a flat gate and an angled trash rack at the inlet. The water level will be controlled by the flat gate using a system for early automatic detection of flood hazards. The tunnel is always filled, thus pressurised. After a flood, the gates at the in- and outlets will close off the tunnel from its environment. The gates are tested twice a year (spring and autumn) and after use, they need to be checked for damages as well. When the gates are tested, the system is flushed and the water is refreshed. The flood bypass tunnel has a discharge capacity of approximately 58 m^3/s. The overview of the location of the flood bypass tunnel is illustrated in Figure 5.23.

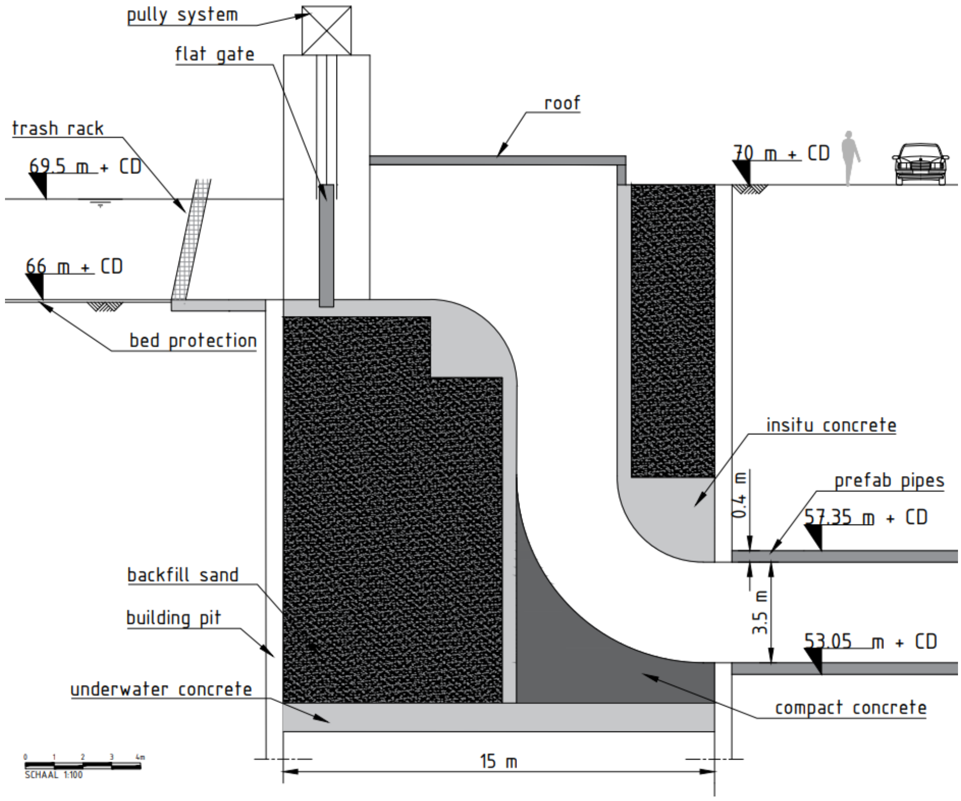


Figure 5.21: Detailed cross-section of the inlet

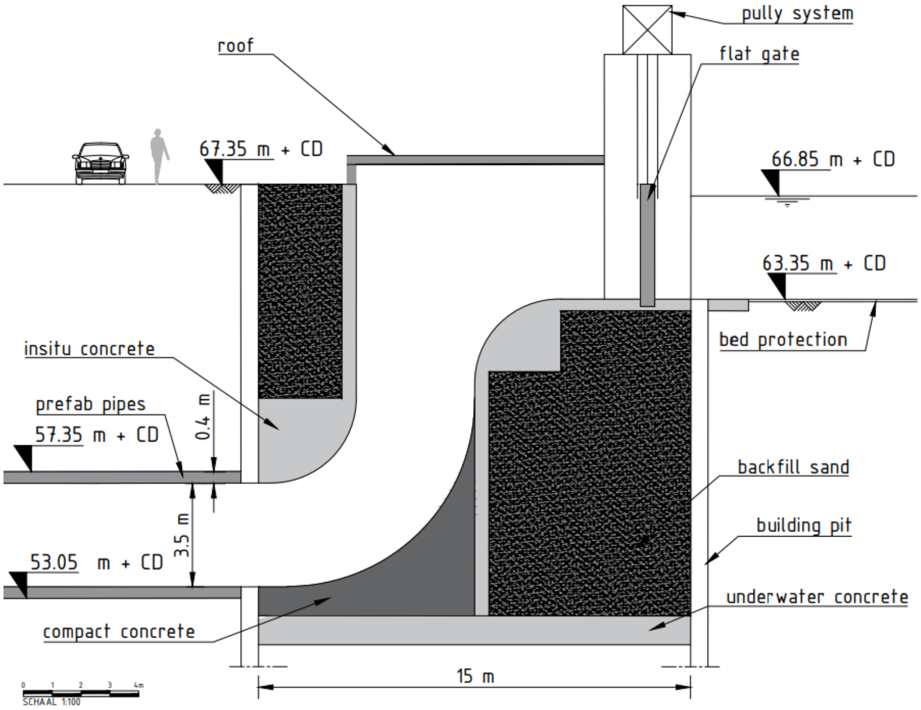


Figure 5.22: Detailed cross-section of the outlet

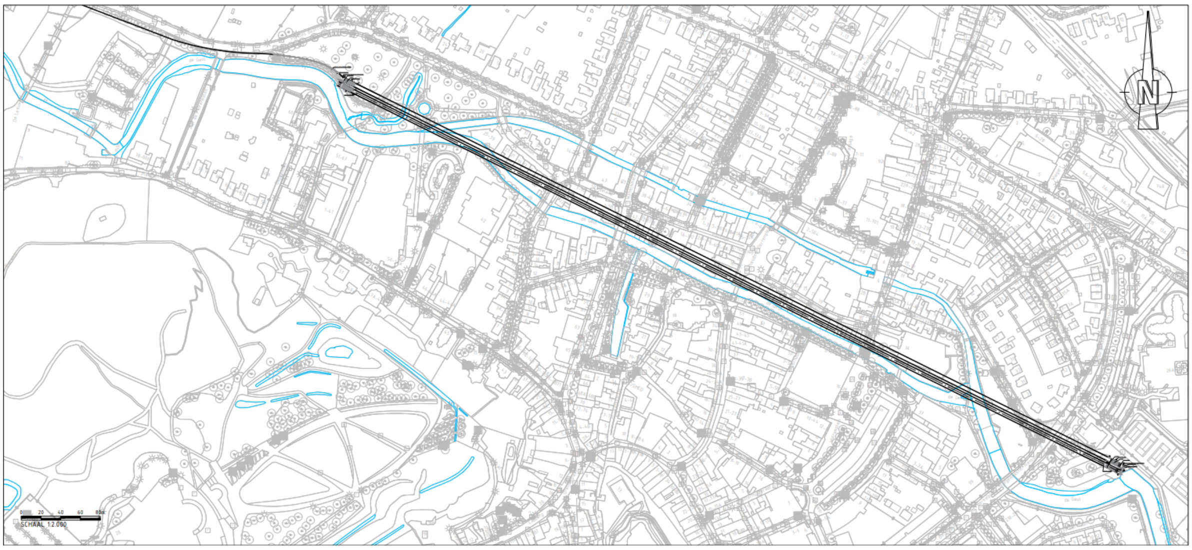


Figure 5.23: Top view of the total design

6

Evaluation of alternatives and selection

The evaluation and selection of alternatives are determined using a multi-criteria analysis (MCA) after which the costs are estimated and compared to the MCA scores of the designs to select the alternative.

The steps of the MCA are as follows determined by Molenaar & Voorendt (2019), see Appendix A:

1. Determine the alternatives;
2. Determine criteria;
3. Determine the weighting of each criterion;
4. Score the alternatives for each criterion;
5. Multiply the score by the weighting for the criterion;
6. Add all the scores for a given alternative and rank the alternatives by their total score.

The first step is elaborated in Chapter 5 and the evaluation criteria are determined in Chapter 3 but are also repeated here, in Section 6.1. Following this the weighting of each criterion is estimated in Section in Section 6.2, after which the alternatives are scored, multiplied and ranked in Section 6.3. The after the scoring the costs are estimated in Section 6.4 and compared to the scores of the designs in the MCA to select the alternative in Section 6.5.

6.1. Evaluation criteria

The evaluation criteria are determined by discussions with the municipality and waterboard.

1. **Effectiveness**;
   The effectiveness of the flood bypass tunnel is determined by the discharge capacity of the flood bypass tunnel during design operation. A design that has a higher discharge capacity during the design operation receives a higher score for this particular criterion.
2. **Reliability**;
   The reliability is determined by the number of steps that have to be taken during the operation, which makes the system more complex. As more steps have a large chance of failure. A design that has a higher reliability receives a higher score for this particular criterion.
3. **Control**;
   The system's control is determined by the degree of regulation over the discharge that flows through the flood bypass tunnel during operation. A design that has a degree of regulation over the discharge receives a higher score for this particular criterion.
4. **Safety**;
   Safety is determined by the limitation of access of civilians to the flood bypass tunnel. A design that has a danger of falling in the flood bypass tunnel lowers the score for this particular criterion.

5. **Maintainability**;
Maintenance ensures that the flood bypass tunnel is operational during a flood. Therefore the structure must be easy to maintain with good access for possible inspections and possible replacements. A structure that is easier to maintain receives a higher score for this particular criterion.
6. **Effect on the environment**;
The effect on the environment is determined by the structure's integration into its surroundings, and the impact the structure has on the environment in terms of nature-friendliness. Therefore, the score for this particular criterion is lower for a stand-alone structure that is not incorporated into its surroundings and is not environmentally friendly.

6.2. Weighting factors of the criteria

The weighting factors of the criteria are determined in discussion with the municipality (Gemeente Valkenburg) and waterboard (Waterschap Limburg) (Gemeente Valkenburg aan de Geul, personal communications, November 28, 2022) & (Waterschap Limburg, personal communications, December 6, 2022). Each criterion is given a score of 1 to 5 based on the importance of the criterion. Less important criterion scores 1 and very important criterion scores 5, see Figure 6.1. The determination of the weighting values is elaborated in Appendix F.1. Reliability, safety and maintainability are valued most according to the municipality and waterboard.

Figure 6.1: Score scale

To weigh the criteria, the sum of the scores of all the criteria is used. The weighting factor (WF) of each criterion is calculated by the score of one criterion divided by the total sum of all the scores of the criteria, followed by a multiplication of 100 to round off the values, see Equation 6.1. The weight per criterion is estimated in Table 6.1.

$$Weightingfactor(WF) = \frac{score}{\sum scores} \cdot 100 \tag{6.1}$$

Table 6.1: Weighting factors of the criteria

	Score (1-5)	WF
1. Effectiveness	3	16
2. Reliability	4	21
3. Control	2	11
4. Safety	5	26
5. Maintainability	4	21
6. Effect on the environment	1	5
$\sum$	19	100

6.3. Evaluation of the design alternatives

The evaluation of the design alternatives is also done by scores. The evaluation is determined in discussion with the waterboard (Waterschap Limburg, personal communications, December 6, 2022). Each design alternative is given a value of 1 to 5 based on the performance of the criterion. A high score means a good value of the design, see Figure 6.1. The evaluation of the design alternatives is elaborated in Appendix F.2.

To weigh the score per design alternative the weighted score is evaluated per criteria after which the sum of the scores of all the criteria per design alternative is used. The weighted score of each criterion is calculated by the score of one criterion times the weighting factor, see Equation 6.2. The total score per design alternative is estimated in Table 6.2.

$$Weightedscore = WF \cdot Score \tag{6.2}$$

Table 6.2: Evaluation of the design alternatives by scores

		Design Alternative 1		Design Alternative 2	
	WF	Score (1-5)	Score · WF	Score (1-5)	Score · WF
1. Effectiveness	16	4	63	3	47
2. Reliability	21	4	84	2	42
3. Control	11	3	32	4	42
4. Safety	26	2	79	3	79
5. Maintainability	21	4	84	3	63
6. Effect on the environment	5	3	16	2	11
$\sum$	100		332		284

The first design alternative obtains a higher score compared to the second design alternative due to its high reliability and maintainability, which are highly valued by the municipality and waterboard.

6.4. Cost analysis

In this section, the costs for each design alternative are estimated. When selecting the preferred design alternative, the cost analysis and the multi-criteria analysis must be taken into account jointly, because a design alternative might have disproportionately high construction and maintenance costs in contrast to the multi-criteria analysis score.

Each design alternative's cost estimation is based on the information retrieved from Witteveen + Bos as the pricing per unit for specific operations or materials is the data that is retrieved. In Appendix G, a detailed description of the cost estimation can be found for each design alternative. The construction costs are described in Appendix G.1, and the maintenance costs are described in Appendix G.2. The total cost estimations for each design alternative can be seen in Table 6.3 and Table 6.4.

Table 6.3: Overview of the estimated construction costs

		Design Alternative 1		Design Alternative 2	
	UNIT PRICE	UNITS	TOTAL COSTS	UNITS	TOTAL COSTS
JACKET (CLOSED FRONT BORE)					
Mobilisation/demobilisation/drilling facilities	€ 1,200,000	1	€ 1,200,000	1	€ 1,200,000
Pipes [/m]	€ 3,500	1700	€ 5,950,000	1700	€ 5,950,000
Pipe jacking [/m]	€ 4,500	1700	€ 7,650,000	1700	€ 7,650,000
SHAFT					
shaft, ≈ 10m deep, 10x15	€ 2,000,000	2	€ 4,000,000	2	€ 4,000,000
Inlet construction passive	€ 1,000,000	1	€ 1,000,000	1	€ 500,000
Outlet construction passive	€ 500,000	1	€ 500,000	1	€ 500,000
GATES	€ 250,000	0	€ 0	4	€ 1,000,000
CONTROL HOUSE MEP	€ 500,000	0	€ 0	2	€ 1,000,000
BED PROTECTION per m^3	€ 50	1050	€ 52,500	700	€ 35,000
DOWNSTREAM					
Co-current channel [/m]	€ 2,000	900	€ 1,800,000	0	€ 0
Widening of the Geul [/m]	€ 1,000	0	€ 0	500	€ 500,000
Replacement of bridges	€ 1,000,000	2	€ 2,000,000	2	€ 2,000,000
TOTAL COSTS			€ 24,152,500		€ 24,335,000
Unforeseen costs (20%)			€ 28,983,000		€ 29,202,000
General Costs, Profit and Risk (AKWR) (18%)			€ 34,199,940		€ 34,458,360
Costs client (VAT) (20%) (engineering/tender)			€ 41,039,928		€ 41,350,032

Table 6.4: Overview of the estimated maintenance costs **per year** with a lifetime of 100 years

			Design Alternative 1		Design Alternative 2	
	UNIT PRICE	**OCCURRENCE**	**UNITS**	**TOTAL COSTS**	**UNITS**	**TOTAL COSTS**
Pumping out	€ 5,000	Every year	1	€ 5,000	0.20	€ 1,000
General maintenance	€ 2,000,000	Once every 25 years	0.04	€ 80,000	0.04	€ 80,000
Replacement of gates	€ 250,000	Once every 40 years 4 gates	0	€ 0	0.10	€ 25,000
Testing of the system	€ 5,000	Twice a year	0	€ 0	2.00	€ 10,000
Electrical & ICT	€ 50,000	Once every 15 years	0	€ 0	0.07	€ 3,333
Mechanical	€ 200,000	Once every 25 years	0	€ 0	0.04	€ 8,000
TOTAL COSTS				€ 85,000		€ 127,333
Unforeseen costs (120% of total)				€ 102,000		€ 152,800

Even though the design alternatives have a similar construction cost, the second design option has higher maintenance costs due to the active gates system. Consequently, the lifetime costs are cheaper for the first design alternative, see Figure 6.3. The life cycle costs should also have been included in the cost estimate, this is discussed in Chapter 8.

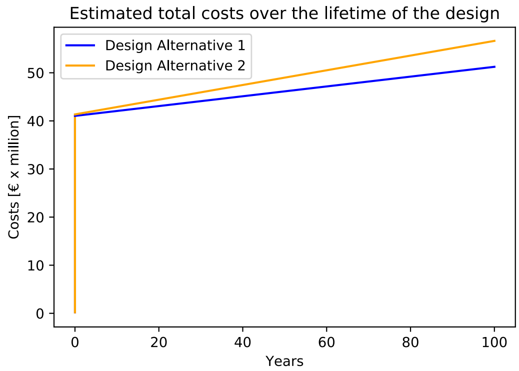


Figure 6.2: Estimated total costs over the lifetime of the design alternative

6.5. Selection of the design alternative

For the selection of the design alternative, The multi-criteria analysis score and the lifetime costs are plotted in Figure 6.3. Following from the graph it can be concluded that the first design alternative is selected because it scores slightly better in both the multi-criteria analysis and cost analysis. This design alternative will be further assessed in Chapter 7. However, even though the selected design alternative scored better in both the cost analysis and the multi-criteria analysis, the difference is marginal. Therefore, both design options may be considered in the future, this is discussed in Chapter 8.

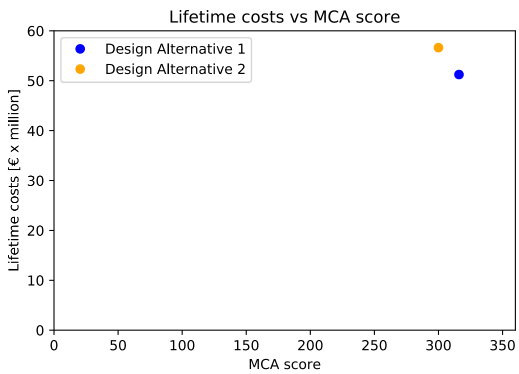


Figure 6.3: MCA score versus total lifetime costs

7
Reduction of the flood risk

In this chapter, the selected design alternative is verified semi-quantitatively on reducing the flood risk in terms of discharge reduction in Valkenburg aan de Geul. This is a simplified analysis as the bifurcation is more complex, involving various parameters which result in different water levels and discharges of the flood bypass tunnel and the Geul which continually change with time. This can be estimated using a model such as SOBEK. This is discussed in Chapter 8.

First, the backwater curves are drawn in various situations to illustrate how the water level in the Geul will approximately respond to the flood bypass tunnel, see Section 7.1. Secondly, the discharges of the flood bypass tunnel and Geul are analysed to form a new discharge-water level curve in Section 7.2. Lastly, the reduction of the risk of flooding in terms of discharge reduction in Valkenburg aan de Geul is semi-quantitatively verified in Section 7.3.

7.1. Water levels in Valkenburg aan de Geul - Backwater curves

As explained in Section 1.2.3, the function of the flood bypass tunnel is to create a temporary bifurcation in the river which will lower the water level locally. This is a simplified analysis as the bifurcation is more complex, involving various parameters which result in different water levels and discharges of the flood bypass tunnel and the Geul which continually change independently in time.

For now, the system is simplified as the incoming discharge Q_{flood} is reduced by discharge through the flood bypass tunnel Q_{FBT} which lowers the remaining discharge through the Geul $Q_{Valkenburg}$. This is based on the simplified Equation 3.1 from Section 3.2.3.

$$Q_{flood} = Q_{Valkenburg} + Q_{FBT}$$

where:				
	Q_{flood}	[m^3/s]	=	discharge of the flood with an estimate return period
	Q_{FBT}	[m^3/s]	=	discharge through the tunnel
	$Q_{Valkenburg}$	[m^3/s]	=	discharge of the Geul

Valkenburg has mild slopes therefore the downstream water level, downstream of the construction is normative. From here, an M1-curve develops upstream, followed by an M2-curve upstream of the inlet until the initial water level is reached again as explained in Section 1.2.3, see Figure 1.2. The co-current channel lets water gradually flow back into the Geul as its cross-sectional area is the same as the flood bypass tunnel's cross-sectional area at the outlet location and its cross-sectional area gradually decreases, due to the difference in water level, see Section 5.1.2. Therefore, the end of the co-channel is indicated as the point where the initial water level is reinstated, thus normative. As the co-current channel gradually decreases its area, the water level from the end of the co-channel upstream will drop slower than expected at the end of the outlet where all the water comes back at once. Based on this, the backwater curves are drawn in various situations to illustrate how the water level in the Geul approximately will respond to the flood bypass tunnel and the co-current channel.

First, for a non-flooding situation, a backwater curve is drawn, after which the backwater curves of the flood situation are drawn. Lastly, a backwater curve is drawn during an extreme flood situation, such as the summer flood of 2021, to look at the ability of the flood bypass tunnel to maintain operations during a crisis (robustness of the design). It is important to note that the figures are not to scale and are meant to schematise the situations. For simplicity, the cross-sectional area of the Geul is assumed the same as in Valkenburg. As this is not the case the water level upstream and downstream appears to be larger than in reality. Also, the bridges and two canals of Valkenburg are simplified to be one simple canal without obstacles.

Non-flooding situation

The boundary conditions of the non-flooding situation are defined in Section 3.2.4. The defined water-level conditions of the non-flooding situation, thus without operation of the tunnel, is a water level at the location of the inlet until CD + 68.4 m and the bottom of the river is located at CD + 66 m. At the location of the outlet, the water level will be in de defined non-flooding situation up until CD + 65.75 m and the bottom of the river is located at CD + 63.35 m. The water depth in a non-flooding situation is 0 to 2.4 m, see Figure 7.1. As the flood bypass tunnel is not in operation, the water level does not change over the length of the Geul.

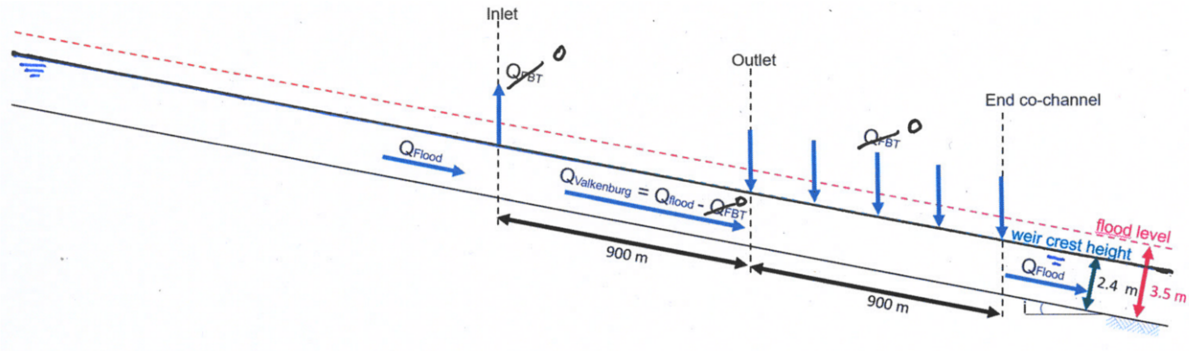


Figure 7.1: Backwater curve non-flooding situation *not to scale*

Flood situation

The boundary conditions of the flooding situation are defined in Section 3.2.4. The defined water-level conditions of the flooding situation, thus with the operation of the flood bypass tunnel, is a water level at the location of the inlet from CD + 68.4 m until CD + 69.5 m and the bottom of the river is located at CD + 66 m. At the location of the outlet, the water level will be in a flood situation up from CD + 65.75 m until CD + 66.85 m and the bottom of the river is located at CD + 63.35 m. The water depth in the defined flood situation is 2.4 to 3.5 m. In Figure 7.2 the backwater curve is drawn. As can be seen in the figure the water level downstream of the construction, is normative. From here, an M1 curve develops upstream, followed by an M2 curve upstream of the inlet until the initial water level is reached again.

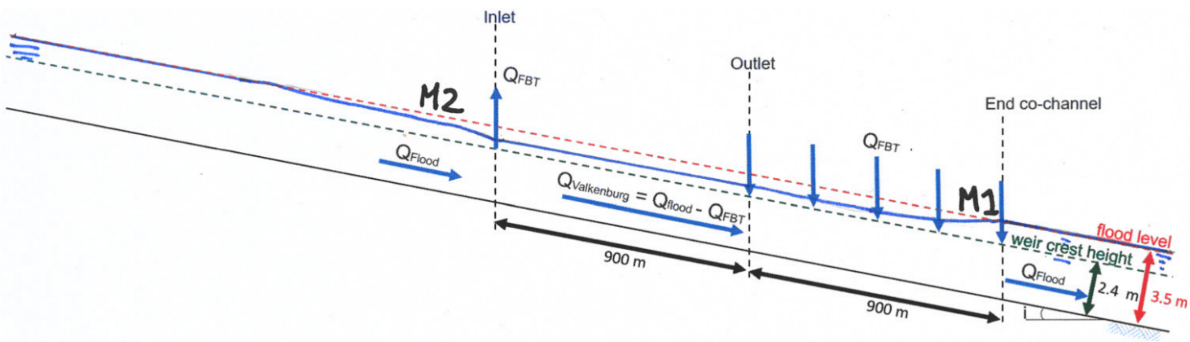


Figure 7.2: Backwater curve flood situation *not to scale*

Extreme flood situation

In extreme flood situations, such as the summer flood of 2021, a flood in Valkenburg aan de Geul cannot be prevented by the designed flood bypass tunnel. As can be seen in Figure 7.3 the backwater curve is drawn the same way as the previous example, however, due to the extreme water level upstream and downstream of the flood bypass tunnel the water level will not drop as much that a flood in Valkenburg can be prevented. If the difference in head remains adequate, the flood bypass tunnel will remain operational during such crises, making the design robust. However, cavitation can occur. This will be elaborated in Section 7.3.

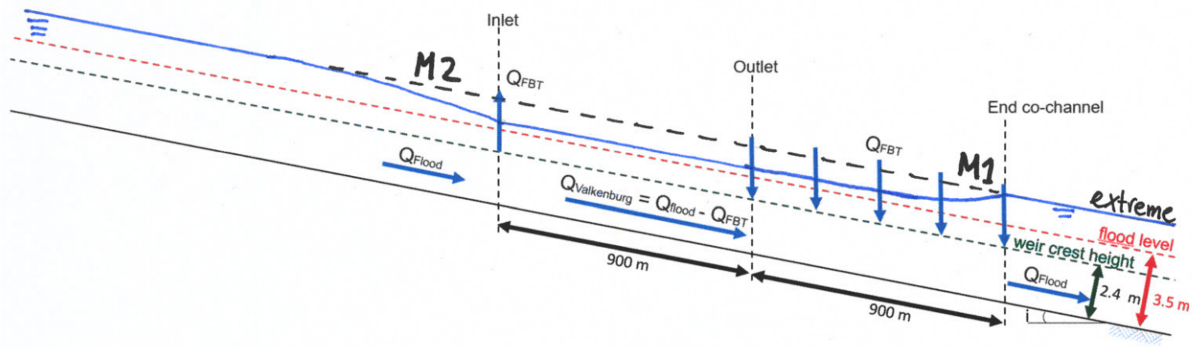


Figure 7.3: Backwater curve extreme flood situation *not to scale*

7.2. Discharges of the flood bypass tunnel and Geul

As analysed in the previous section, the potential water level changes are caused by the difference in discharge through the flood bypass tunnel and the Geul. To simplify the discharge analysis, the discharge in the flood bypass tunnel is based on the discharge over the Ogee weir only. Therefore, its only parameter is the upstream water level. The difference in head through the tunnel is of course important as well, but for now, it is assumed that there is enough of the difference in head (2.65 m) to make this variable independent.

First, the discharges of the flood bypass tunnel are analysed based on the assumption that the difference in head is sufficient. Secondly, the discharge and water level of the Geul at the inlet location are analysed, which results in a new discharge-water level curve. Lastly, an extreme flood situation, such as the summer flood of 2021, is analysed to assess the robustness of the design.

Discharges of the flood bypass tunnel

The discharges of the flood bypass tunnel are analysed using the discharge over the Ogee weir formula of Equation 5.2, with the assumption that the difference in head is sufficient, thus making this variable independent. Therefore assuming that the discharge through the tunnel is only based on the water level at the inlet location until the water level it is designed for. In Figure 7.4 the discharge over the Ogee weir is given. For a water level at the inlet of CD + 66 m until CD + 68.4 m the tunnel will not be in operation, thus the discharge will be zero. For a water level of CD + 68.4 m, the discharge will flow over the weir into the flood bypass tunnel. Here Equation 5.2 is plotted. For the energy level H, the water level at the inlet just before the Ogee weir is used, therefore the maximal energy level is CD + 69.38 m due to the head loss of the trash rack.

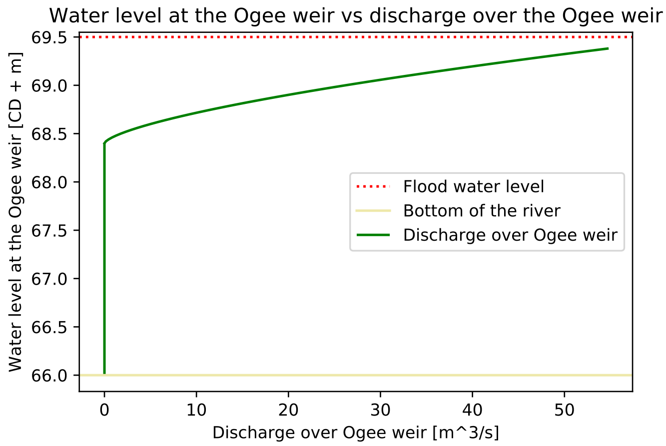


Figure 7.4: Water level at the Ogee weir vs discharge over the Ogee weir

Discharges and water level of the Geul

Due to the fact that the Geul in Valkenburg has a rectangular cross-section, it is assumed that the current discharge-water level curve is linear. The curve is plotted based on the points that the water level at the bottom of the river correspondents with a discharge of 0 m^3/s and that the flood water level in the current situation correspondents with a flood discharge of 47 m^3/s, see Section 3.2.3 (Deuss et al., 2016). The current discharge-water level curve is plotted in blue in Figure 7.5.

Adding the discharge over the Ogee weir curve of Figure 7.4 to the current discharge-water level curve results in the estimated new discharge-water level curve. This is plotted in green in Figure 7.5. It can be assessed that the water level is reduced for a higher discharge which follows from the backwater curves of Section 7.1. It follows that for a water level of CD + 69.5 m, 97 m^3/s can be discharged. With 47 m^3/s through the Geul and 50 m^3/s through the flood bypass tunnel with sufficient difference in head (2.65 m). What can be analysed as well is that the tunnel starts to flow with approximately 32 m^3/s, as indicated by the black marker.

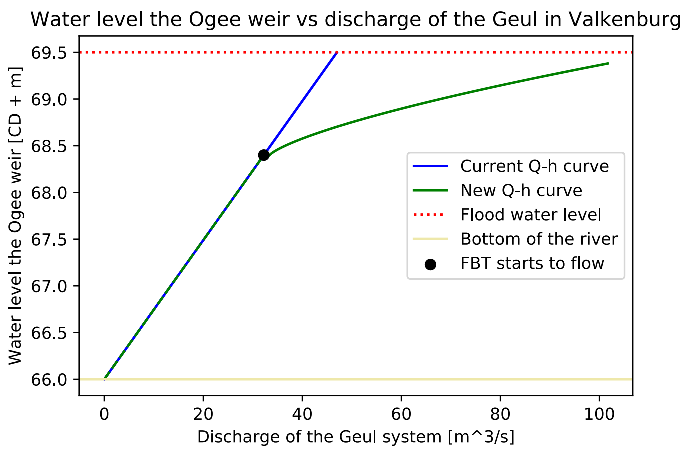


Figure 7.5: Q-h curve of the Geul in Valkenburg

Discharges of the flood bypass tunnel in an extreme flood situation

The discharges of the flood bypass tunnel in an extreme flood situation, such as the summer flood of 2021, are analysed using the discharge over the Ogee weir formula of Equation 5.2 of Section 5.1.2. Assuming that the difference in head is greater than 2.65 m and the Geul's water level is greater than

CD + 69.5 m. Also, assuming that the discharges over the weir are only affected by water level (and that the weir is not submerged). This discharge is depicted as a dotted green line in Figure 7.6.

The discharge over the weir is possible without the risk of cavitation damage until 130% of the energy level (US Army Corps of Engineers, 2022). Therefore, for roughly a water level of 1.27 m (this corresponds to CD + 69.67 m), see Equation 5.3, cavitation will occur and the weir will be damaged, see the red line in Figure 7.6. The flood bypass tunnel maintains its operations until this point. As cavitation occurs the system becomes unreliable and will not be robust. This would be a very extreme situation with an extreme difference in head, extreme flooding of Valkenburg, and no submergence of the weir, which is a rare combination.

The most likely limiting factor however is the difference in head. The difference in head limits the discharge through the flood bypass tunnel. Therefore, the discharge over the weir will be larger than the discharge through the flood bypass tunnel. This results in an increase in water level upstream and therefore submerged flow conditions at the weir. This is further discussed in Chapter 8, and recommendations for further research for the submergence are done in Section 9.2.

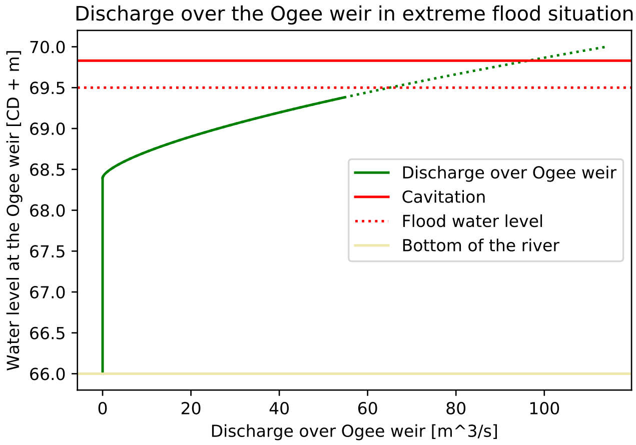


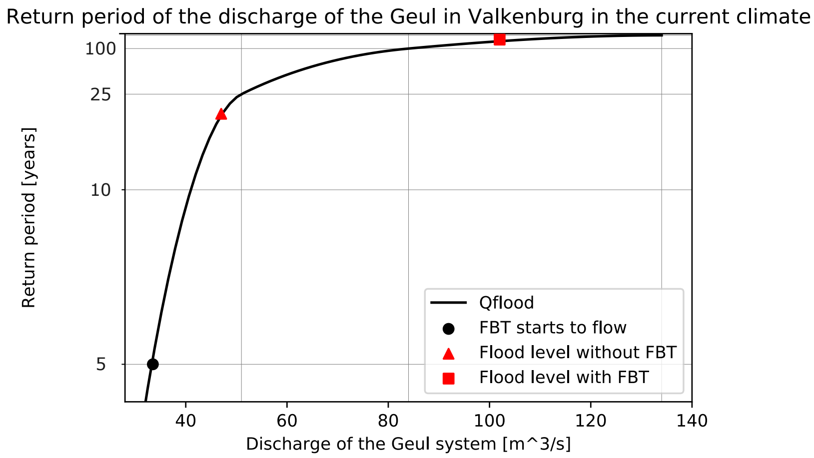
Figure 7.6: Discharge over the Ogee weir in extreme flood situation

7.3. Reduction of the flood risk

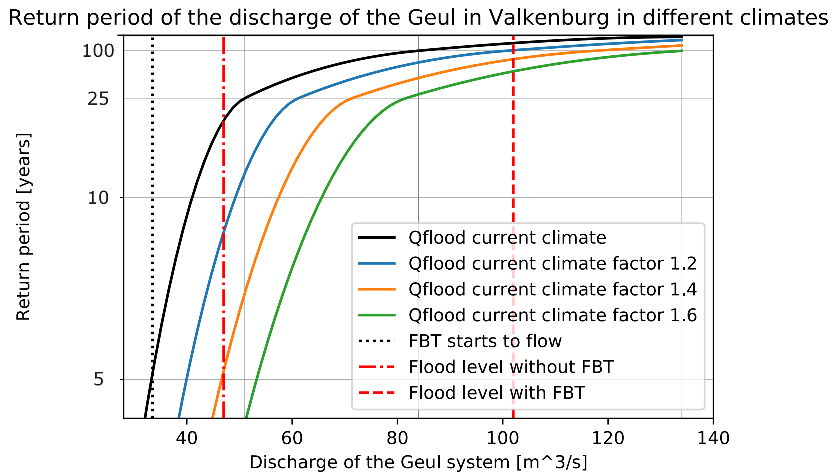
"Risk is the probability of a flood event multiplied by the consequences" (Jonkman et al., 2018). Therefore, assuming that the consequences do not change the reduction of the probability of a flood for Valkenburg leads to a flood risk reduction. The reduction of the flood risk in terms of discharge reduction in Valkenburg aan de Geul is semi-quantitative verified in Figure 7.7a.

The current discharge capacity of the Geul in Valkenburg of 47 m^3/s correspondents to a flood once every 19 years. Due to the flood bypass tunnel, an additional discharge capacity of 55 m^3/s has been applied, this results in a maximal discharge capacity of 102 m^3/s for the Geul's system in Valkenburg. This discharge corresponds to a return period of once every 250 years. This reduces the flood risk based on the discharge from an estimated once every 19 years to once every 250 years in the current climate. Figure 7.7b gives the reduction of the risk of flooding in different climate scenarios.

The flood bypass tunnel tunnel starts to flow at a discharge of approximately 32 m^3/s, therefore from the figures it is analysed that this has a return period of once every 5 years.



(a) Return period of the discharge of the Geul in Valkenburg in the current climate



(b) Return period of the discharge of the Geul in Valkenburg in different climates

Figure 7.7: Return period of the discharge in Valkenburg aan de Geul

The return periods of the discharge through Valkenburg's city centre in the case with and without a flood bypass tunnel are plotted in Figure 7.8. The current discharge capacity of the Geul through the city centre of Valkenburg is 47 m^3/s, which corresponds to a flood once every 19 years. The discharge capacity of the Geul through the city centre of Valkenburg remains the same with a flood bypass tunnel, however, the return period corresponds now to a flood once every 250 years. This reduces the flood risk based on the discharge from an estimated once every 19 years to once every 250 years in the current climate. When the Geul through Valkenburg floods, with or without the flood bypass tunnel, the graphs will run parallel for high return periods. This is not very clear from the graph as the maximum return period of this graph corresponds to the summer flood of 2021.

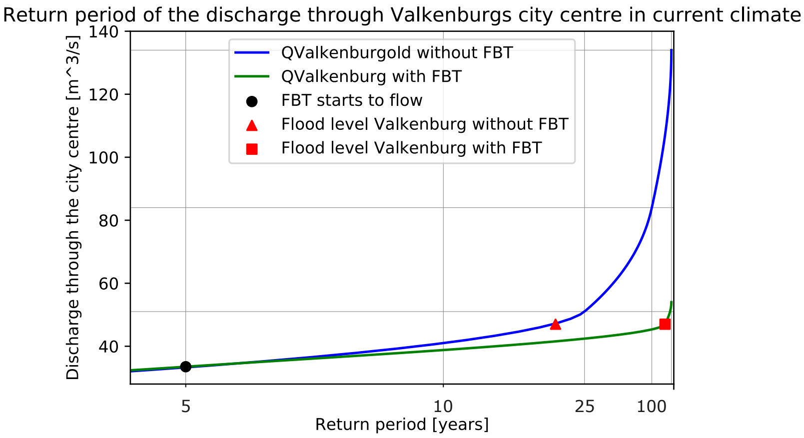


Figure 7.8: Return period of the discharge through Valkenburg's city centre in current climate

From Figure 7.8, the discharge through the city centre of Valkenburg aan de Geul can be coupled with the return period, which could be plotted against the consequences (damage). The difference between the numerical integration of these two graphs lead to the estimated reduction in flood risk. The addition of the consequences is recommended for future research by means of a cost-benefit analysis. This is further discussed in Section 9.2.

8 Discussion

During the design process, numerous considerations had an effect on the final design. This chapter discusses the variables that affected the design process's outcomes.

Unavailability of boundary conditions and its statistics
Due to the unavailability of insights into (boundary) conditions and their statistics, the design is based on a variety of assumptions.

The return periods of the water levels are unavailable, and water level measurements are not made precisely at the inlet and outflow locations, but nearby; hence, the water level, river bottom level, and thus the difference in head may vary. Which impacts the resulting design and discharge capacity of the flood bypass tunnel.

The design is also based on assumed discharge statistics. Due to the absence of measurement, the discharges and their precise recurrence period are uncertain, as the discharge statistics for the Geul are unavailable. Consequently, the estimated reduction in the risk of flooding in Valkenburg aan de Geul may vary, resulting in different boundary conditions for this design.

In addition, the design is based on the available cone penetration test (CPT), which indicates that the initial soil layers consist of relatively soft soils (clay/sand) and underneath that a layer of limestone. In this situation, pipe jacking is possible and a shallow foundation can be used. However, while a variety of CPTs are available, their results vary and are not conducted at the flood bypass tunnel's precise location. This creates uncertainty regarding the soil conditions.

Deep uncertainty regarding climate change statistics
Climate change accelerates the recurrence of floods; however, the precise impact of this on the return periods of the discharges and water levels of the Geul is unknown due to its deep uncertainty. As no prior research has been conducted on the future projection for the Geul, the following scenarios have been developed based on the existing literature and underlying assumptions and are presented in Appendix C.2. Based on this, it is assumed that the discharge corresponding to the current 25-year return period is shifting towards a return period of 100 years. Therefore the discharge corresponding to the 100-year safety level is used as a boundary condition in the design. This is to ensure the safety standard.

Downstream and environmental effects
The thesis is focused on reducing the flood risk in Valkenburg aan de Geul which means that upstream and downstream locations are not incorporated into the design. Although the author is aware that a greater or quicker discharge might have implications for locations further downstream. The recently published research by Asselman & van Heeringen (2023) concluded that replacing bridges and increasing the discharge capacity of Valkenburg aan de Geul, for example by the operation of a flood bypass tunnel, decreases the water level significantly, but passes the effect on downstream by increasing the water level by 1 to 2 cm. Therefore a combination of flood reduction measures, such as an additional

culvert beneath the Julianakanaal and additional retention areas further upstream and downstream of Valkenburg, would likely be the most effective way to reduce the risk of flooding, this is outside the scope of the study.

The flood bypass tunnel also has an influence on the environmental effects of the Geul. However, because the flood bypass tunnel is only temporarily in use and is only expected to operate once every five years, the effect is only expected to be temporary, and the system will revert to its prior form once the sediments are pumped back into the system. Therefore the morphological response is anticipated to be moderate. However, the impact of other environmental effects such as water quality, nitrogen deposition around the Natura 2000 area, carbon dioxide emissions and the effect on ecosystems has not been studied.

Design method

The approach to reach the objective was to combine a design approach for hydraulic structures with a Systems Engineering approach. The design approach was useful for this type of design, however, a flood bypass tunnel has never been applied in the Netherlands as a flood measure and there were not many resources available for its application for Valkenburg aan de Geul. Because of this, the reference projects played a more significant role in this research than indicated by the method. The Systems Engineering method was useful to organise the design process of the flood bypass tunnel into sub-systems and into elements. As the inlet, the tunnel and the outlet were evaluated separately, this way it was easier to cover the operation, maintenance, hydraulic design, and costs.

Reference projects

In the design method, the reference projects were used as starting point. They were chosen because of their differences and similarities in characteristics which makes them applicable to this situation. However, there are more examples than these. On top of this, not all the data regarding the reference projects are available.

The reference projects differ from the proposed Valkenburg tunnel because the pipe jacking technique will be utilised, which requires a maximum diameter of 3.5 meters, while the reference flood bypass tunnels are drilled tunnels with a diameter of four metres or more, and these projects had a larger difference in head. Nevertheless, the reference projects are relevant due to their similarity in shape, and application to minimise energy loss and pressure peaks by pressurised flow, and operation. In this way, the designed flood bypass tunnel is not unique, in terms of design and operation. However, its application in the Netherlands and the combination with the cost-efficient pipe jacking method is unique.

Development and verification of the flood bypass tunnel design alternatives

Currently, the location of the tunnel track is assumed to be a straight line between the two parking lots to reduce hydraulic head loss due to the lack of bends. However, it may be legally more convenient to have the tunnel track remain under the property of the water board as much as possible, by shifting the tunnel or creating a slight curve in the route so that you end up under the Geul. In addition, the municipality indicated that a care facility is located upstream of the inlet and that an additional area might be developed there.

In this thesis, the pipe jacking method is applied as it is a cost-efficient boring method. Since the cost of construction per meter is relatively constant for different diameters while the discharge reduces greatly for a smaller diameter, the flood bypass tunnel design assumed a double tunnel with the maximum diameter possible for this method, which is 3.5 m. However, the number of tunnels and diameter could be optimised with a cost-benefit analysis. This is not covered in this thesis.

As the design alternatives were based on the reference projects the Ogee weir and flat gates were chosen for these design alternatives as they proved to apply to the reference projects. However, when further developing the design alternatives, a different type of weir, e.g., a zigzag weir, or a different type of gate, e.g., a flap gate, might turn out to be more suitable for this application if this is preferred by the stakeholders. Covering the weir with a roof might be safer and could serve a multi-functional purpose.

In this thesis, some opportunities were scoped out to focus on the hydraulic flood bypass tunnel design and operation which reduce the flood risk in Valkenburg aan de Geul. These opportunities can be looked at in future research. In Appendix H add-on opportunities have been looked at, e.g., retention area upstream, adding barriers in the Geul, using sewage pipe, and usage of different types of gates. Adding a barrier in the Geul together with a retention area upstream could also guarantee a significant difference in head to prevent submergence of the weir.

In Appendix I sustainable opportunities have been looked at as well. However, these fell outside the scope of this thesis. Adding these would have changed the design and added combinations, which might be interesting to explore these opportunities in further research.

In the hydraulic design, numerous head loss coefficient assumptions were made. As there is a range of head loss coefficients possible, a Computational Fluid Dynamics (CFD) model of the flood bypass tunnel's inlet and outlet could be developed to determine the head losses more accurately. This results in a more reliable discharge capacity of the designed flood bypass tunnel.

Evaluation and selection interpretation
The evaluation criteria, weighting, and evaluation of the alternatives could vary based on the individual. To prevent this from happening in this analysis, multiple stakeholders were included (Waterschap Limburg and Gemeente Valkenburg aan de Geul), so the interpretation would only vary marginally. The cost estimation is based on highly approximate numbers due to the uncertain exact structural design and the variable building costs and inflation. The life cycle costs are also neglected. Therefore, even though the selected design alternative scored better in both the cost analysis and the multi-criteria analysis, the difference is marginal. Consequently, both design options may be considered in future designs depending on the real costs and the variance in preferred characteristics of all the relevant stakeholders.

Simplification of the flood risk system
In Chapter 7 a simplified analysis of the flood risk system was performed. It demonstrated that, based on the discharge, the proposed flood bypass tunnel reduces the flood risk from an estimated once every 19 years to once every 250 years in the current climate. However, the flood bypass tunnel and the Geul have continuously fluctuating water levels and discharges as the bifurcation is more complex and incorporates multiple parameters. Part of the analysis evaluates the discharges and water levels over the Ogee weir based on the assumption that the difference in head is sufficient and that the Ogee weir is not submerged, which may not be the case and reduce the discharge capacity significantly.

9 Conclusion and recommendations

This Chapter consists of the thesis's conclusion and recommendations. The conclusion of this thesis is described in Section 9.1. Section 9.2 elaborates on the most important recommendations that followed from the discussion in Chapter 8.

9.1. Conclusion

A flood bypass tunnel has never been applied in the Netherlands, and there is no prior research detailing the hydraulic and operational applicability of a flood bypass tunnel for Valkenburg aan de Geul. The main objective of this thesis is therefore to develop a hydraulic flood bypass tunnel design for Valkenburg aan de Geul which should operate to reduce the flood risk.

In this project, two design alternatives were considered: an active and a passive flood bypass tunnel. The passive flood bypass tunnel is selected due to its reliability and maintainability. It consists of a 2.4-meter-tall and 24.5-meter-wide Ogee weir, two tunnel tubes of 3.5 m diameter, and allows for a maximum discharge capacity of 55 m^3/s. The proposed flood bypass tunnel was designed to reduce the flood risk based on discharge reduction from an estimated once every 19 years to once every 250 years in the current climate. This accounts for uncertainty due to climate change and ensures flood risk reduction in the future. The flood bypass is only active when a flood is impending; hence the water continues to flow through the Geul and does not interfere with the cultural heritage and tourism of Valkenburg aan de Geul. After a flood, the water in the flood bypass tunnel will be pumped out. The design is shown in Figures 9.1, 9.2 and 9.3, and as an attachment to the thesis.

Recent research did not look further into the opportunity of a flood bypass tunnel as a flood measure assuming it would be too expensive. However, due to the cost-efficient pipe jacking method, the total construction cost is approximately €40 million with an estimated yearly maintenance cost of €100 k.

Overall, the project aimed at answering the following research question:
"What is the hydraulic design of a flood bypass tunnel to reduce the risk of flooding in Valkenburg aan de Geul?"
The research question is answered through the following five sub-questions.

Sub-question 1: "What is the current situation of the river system and environment of Valkenburg aan de Geul?"
The area of Valkenburg aan de Geul has limited space, including restrictions to protect the culture and nature, which makes developing a flood reduction measure challenging. The Geul's river system is heavily dependent on precipitation; therefore, the average discharge is very low (1-4 m^3/s), but the larger return periods are characterised by a large increase of the discharge, which is typical for smaller catchment areas in mountainous environments. The discharge is estimated to increase in the future due to climate change.

Sub-question 2: ”What can be learnt and/or applied from reference projects?”
From the reference projects, it can be learnt that there are two different types of flood bypass tunnels: passive and active, see Table 4.2. For a passive flood bypass tunnel, such as those in Dallas and San Antonio, an Ogee weir inlet can be applied as well as after operating, the water in the flood bypass tunnel can be pumped out. For an active flood bypass tunnel, such as those in Lyss, Sarneraa and Thun, a vertically moving flat gate at the inlet and the outlet can be applied. Also, the operation of a system for early automatic detection of flood hazards can be applied. Furthermore, the flood bypass tunnel is filled and closed off when not in operation. However, for maintenance, the water in the tunnel is pumped out.

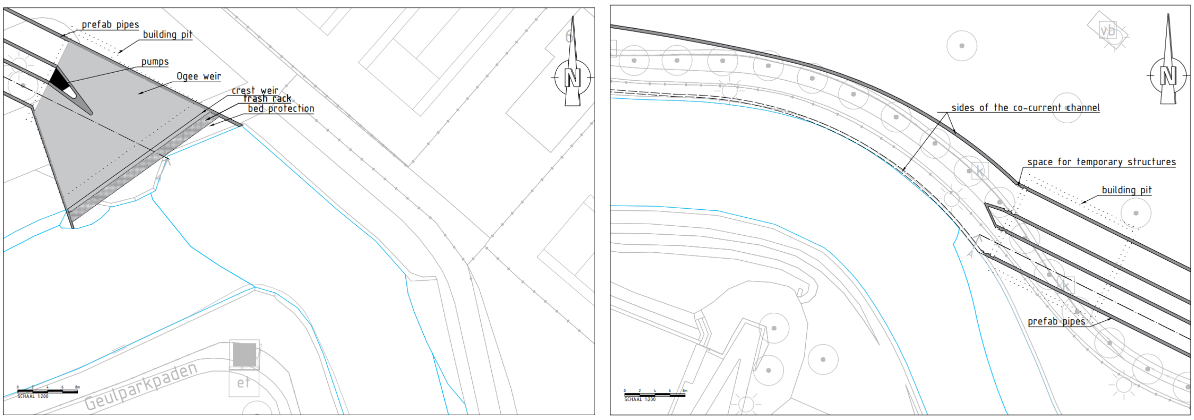
Sub-question 3: ”What are possible alternatives for the design of a flood bypass tunnel in Valkenburg aan de Geul?”
Based on the reference projects, two design alternatives were considered, a passive and an active flood bypass tunnel.

The first design alternative is based on a passive flood bypass tunnel. It consists of a 2.4-meter-tall and 24.5-meter-wide Ogee weir, two tunnel tubes of 3.5 m diameter, and allows for a maximum discharge capacity of 55 m^3/s. Once the tunnel is filled, the flow is pressurised. After a flood, the remaining water will be pumped out. A co-current channel is designed to prevent water from refilling the tunnel from the outlet side in a non-flooding situation. This channel will flow parallel to the Geul and gradually discharges water back into the Geul due to the difference in water level.

The second design alternative is based on an active flood bypass tunnel. It consists of four vertical moving flat gates at the inlets and outlets, two tubes of 3.5 m diameter, and allows for a maximum discharge capacity of 58 m^3/s. The water level will be controlled by the flat gates using a system for early automatic detection of flood hazards. The tunnel is always filled, thus pressurised. After a flood, the gates at the in- and outlets will close off the tunnel from its environment. The gates are tested twice a year, during which the system is flushed and refreshed.

Sub-question 4: ”What flood bypass tunnel design alternative is selected based on the evaluation of the alternatives?”
The two design alternatives were evaluated according to six weighted criteria and ranked using a multi-criteria analysis in discussion with the municipality (gemeente Valkenburg aan de Geul) and the waterboard (waterschap Limburg), and a cost analysis was conducted. The first design alternative, with the passive flood bypass tunnel, was selected due to its high reliability and serviceability, which were highly valued, and low maintenance costs, compared to the second design alternative.

Sub-question 5: ”To what extent does the selected design alternative semi-quantitatively reduces the risk of flooding in Valkenburg aan de Geul?”
The selected design alternative semi-quantitatively increased the discharge capacity in Valkenburg aan de Geul with a 55 m^3/s discharge capacity. Together with the Geul's current discharge capacity in Valkenburg of 47 m^3/s, this results in a maximal discharge capacity of 102 m^3/s for the Geul's system in Valkenburg. This reduces the flood risk based on the discharge from an estimated once every 19 years to once every 250 years in the current climate.



(a) Top view of the inlet

(b) Top view of the outlet

Figure 9.1: Top view of the inlet and outlet

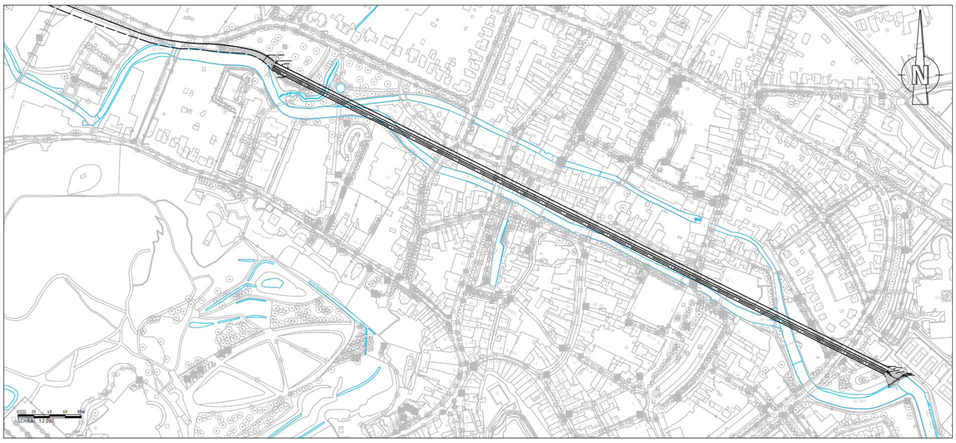
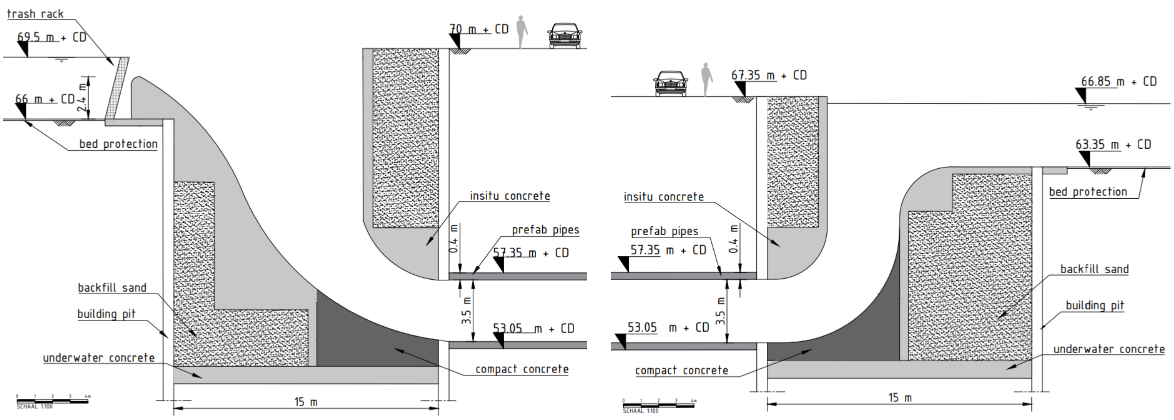


Figure 9.2: Top view of the total design



(a) Cross-section of the inlet

(b) Cross-section of the outlet

Figure 9.3: Cross-sections of the inlet and outlet

9.2. Recommendations

As described in the Discussion in Chapter 8, the following recommendations are advised to be implemented in further research:

Accurate data and statistics

More accurate data and statistics are recommended for future research. This consists of:

- More accurate cone penetration tests at the flood bypass tunnel's precise location to make sure the pipe jacking is possible and a shallow foundation can be used.
- More discharge measurement stations are necessary in flood-safe locations to get accurate statistics on the return periods with discharges of the Geul in Valkenburg.
- Discharge measurement is necessary to measure the water level at the inlet, outlet, and end of the co-current channel to get accurate statistics on the return periods with water levels at the needed locations.
- More research is needed to study the return periods of the discharges and water levels of the Geul in the future to avoid oversized design.

SOBEK model

The bifurcation is very complex and incorporates multiple parameters. The flood bypass tunnel and the Geul have continuously fluctuating water levels and discharges, which affects the difference in head in the flood bypass tunnel and the potential submergence of the Ogee weir, which reduces the discharge capacity. It is recommended to make an approximation of this using a SOBEK model. Following this, a detailed calculation of the risk reduction can be done and the reduction of the risk can be estimated more accurately. This would involve evaluating what the impact of the flood bypass tunnel is in terms of quicker discharge for locations further downstream. It would also be interesting to study the effect of a combination of flood reduction measures, such as an additional culvert beneath the Julianakanaal and additional retention areas further upstream and downstream of Valkenburg using a SOBEK model, as this would likely be the most effective way to reduce the risk of flooding.

Computational fluid dynamics model

To get a more accurate estimate of the flood bypass tunnel's head loss, thus discharge capacity, a CFD model of the tunnel's inlet and outlet could be developed. This results in a more reliable discharge capacity of the designed flood bypass tunnel.

Cost-benefit analysis

To investigate the optimal position and length of the tunnel track and/or the optimal number of tunnels and diameter, a cost-benefit analysis would be recommended. The analysis should incorporate a detailed calculation of the flood risk reduction and include the impact of broader societal and environmental effects such as water quality, nitrogen deposition around the Natura 2000 area, carbon dioxide emissions and the effect on ecosystems.

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A
Methods

This appendix elaborates on the methods used to reach the objective. In Section A.1 the design approach for hydraulic structures is explained. In Section A.2 the Systems Engineering approach is explained and in Section A.3 the multi-criteria analysis is explained.

A.1. Design approach for hydraulic structures

Two design approaches are considered the basic design approach, see Figure A.1, and the integrated design approach, see Figure A.2. As the integrated design approach follows from the basic design approach the steps are similar (M. M. Voorendt & Vakgroep Waterbouwkunde (Delft), 2017). The main difference is that the integrated design, the approach takes into account the environment and society as well as the realisation, operation and second life.

As every hydraulic structure design differs, the design approach is a combination of both approaches. The phases are defined in chapter 1.

A.2. Systems Engineering approach

The Systems Engineering approach is used to organise the design process of a complex project and/or large project. This is done by splitting the overall system into smaller sub-systems which makes it easier to design. Using this approach it is easier to cover the whole life cycle while considering the environment, cost and benefits, design & development, operation and maintenance, etc (Molenaar & Voorendt, 2019). In Figure 1.7 the system is organised for the flood prevention of Valkenburg.

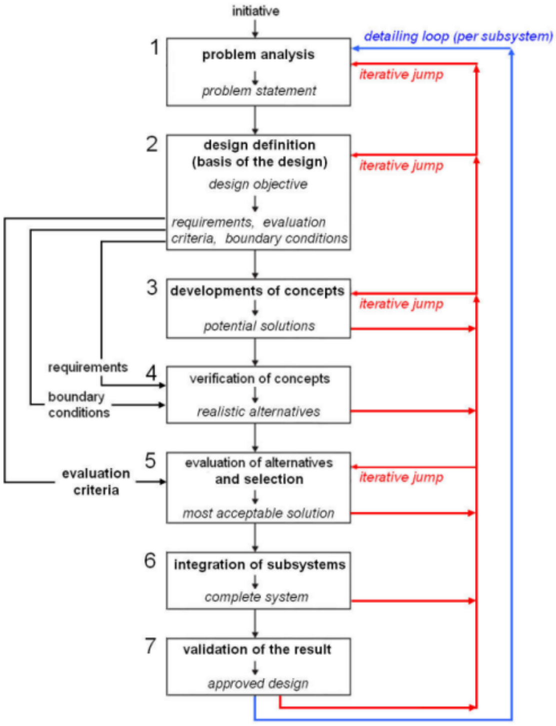


Figure A.1: Basic design approach(Molenaar & Voorendt, 2019)

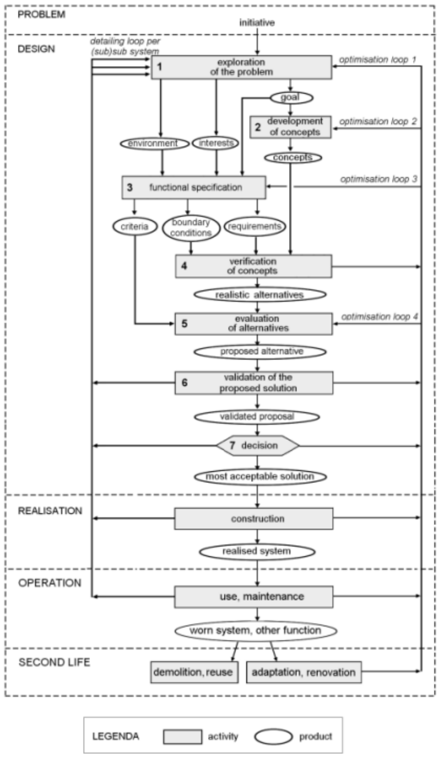


Figure A.2: Integrated design approach (M. M. Voorendt & Vakgroep Waterbouwkunde (Delft), 2017)

A.3. Multi-criteria analysis (MCA)

The multi-criteria analysis is a method by which alternatives can be compared using different qualitative and quantitative criteria. Criteria like costs and benefits are measured in euros (quantitative), whilst environmental impacts can only be measured in a relative way (qualitative). This complicates the comparison of the alternatives in choosing the best alternative (Molenaar & Voorendt, 2019).

The steps are as follows by Molenaar & Voorendt (2019):

1. Determine the alternatives;
2. Determine criteria;
3. Determine the weighting of each criterion;
4. Score the alternatives for each criterion;
5. Multiply the score by the weighting for the criterion;
6. Add all the scores for a given alternative and rank the alternatives by their total score.

Some important notes are that requirements should not be formulated as evaluation criteria, because all alternatives should already satisfy all requirements before they can be evaluated. The other important note is that costs should not be formulated as a criterion. The best design has the highest value-cost ratio (Molenaar & Voorendt, 2019). See Chapter 6 for the evaluation and selection of the alternatives based on the multi-criteria analysis and the cost analysis.

B
System analysis

This appendix provides extra information to the system analysis of Chapter 2. In Section B.1 the area analysis is supported. Section B.2 consists of the stakeholder analysis and Section B.3 consists of the soil profile and geological surveys.

B.1. Area analysis

This section provides extra information for the area analysis. In Figure B.1 an overview of cultural heritage is illustrated. In Figure B.2 an overview of Natura 2000 area is illustrated and in Figure B.3 the inundation area upstream of Valkenburg is illustrated.

Figure B.1: Overview of cultural heritage (Rijksdienst voor het Cultureel Erfgoed, 2022)

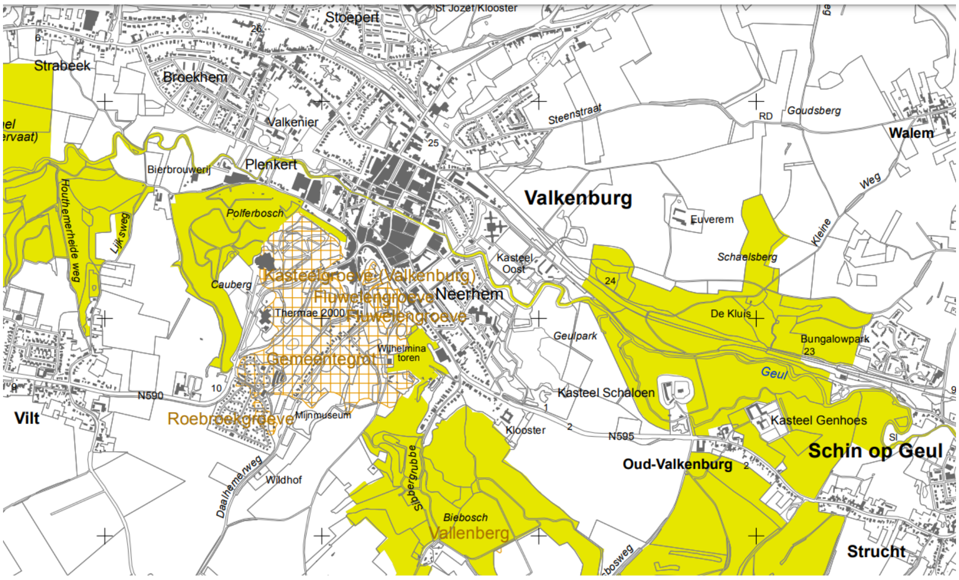


Figure B.2: Overview of Natura 2000 area (Natura 2000, 2000)

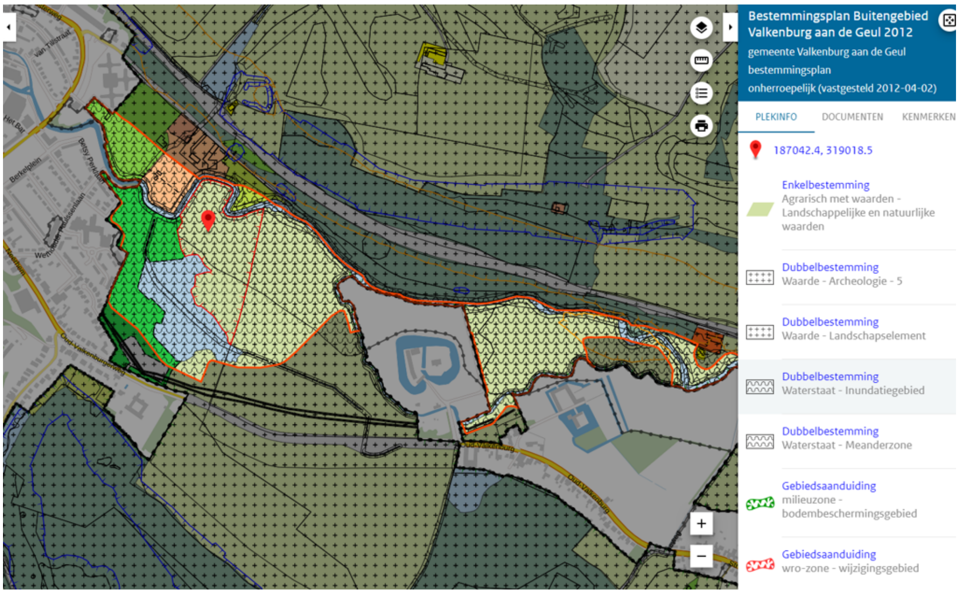


Figure B.3: Inundation area upstream of Valkenburg (Ruimtelijke plannen, 2022)

B.2. Stakeholders analysis

Figure B.4 depicts the stakeholders analysis

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

Figure B.4: Stakeholders analysis (Kallen et al., 2022)

B.3. Soil

This section consists of the soil profile and geological surveys.

Soil profile

The soil profile is displayed in Figure B.5. The soil profile consists of the following two layers:

0.00 m - 3.50 m
Lithology: Complex layer, consisting of a variation of sandy clay, medium and fine sand, clay and peat and a little coarse sand.
Hydrogeology: Complex layer of Holocene deposits.

3.50 m - 25.00 m
Lithology: Limestone layer, consisting of limestone with a few flint banks.
Hydrogeology: Maastricht Formation, limestone layer (outside the mapping area, not verified).

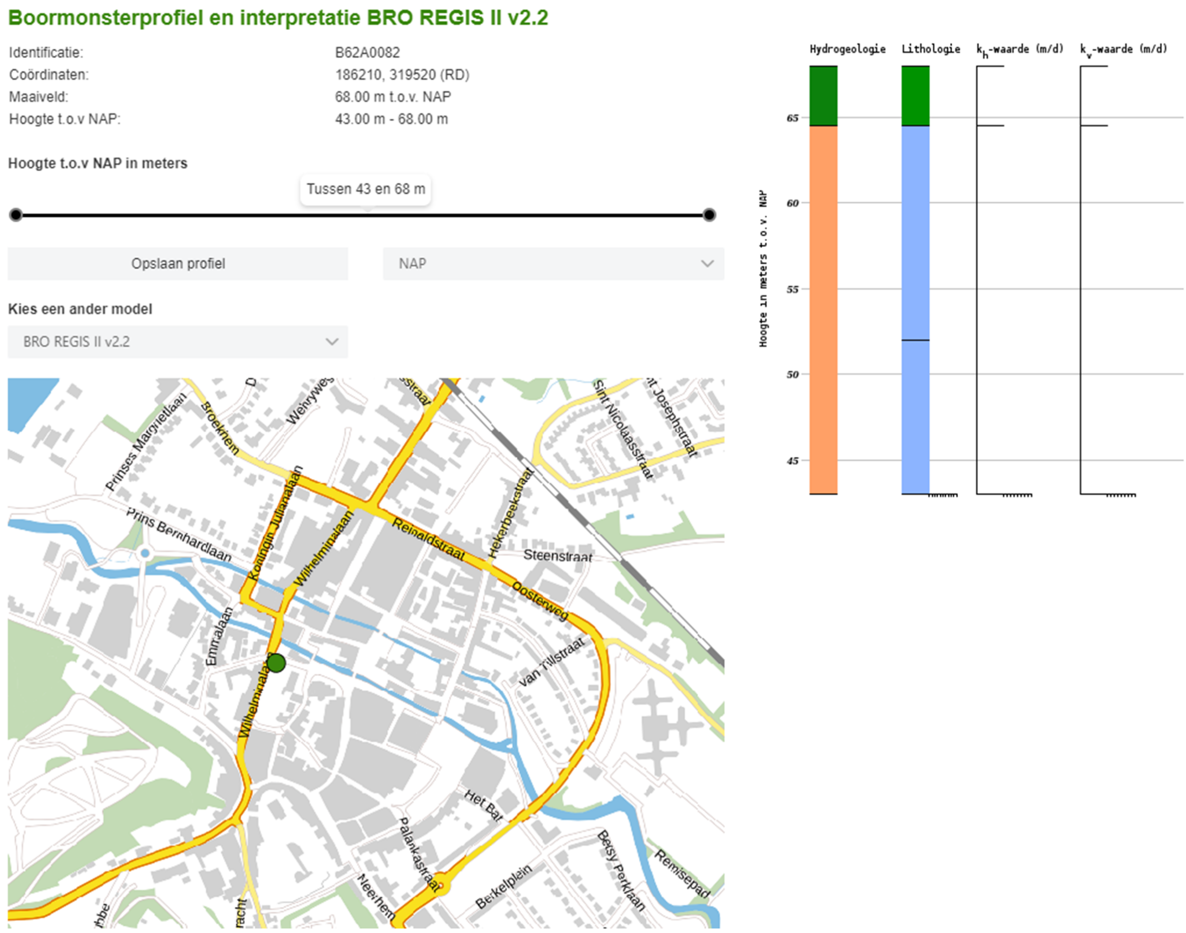


Figure B.5: Soil profile (TNO Geologische Dienst Nederland, 2022)

Geological survey

This section depicts the geological surveys in Valkenburg aan de Geul.

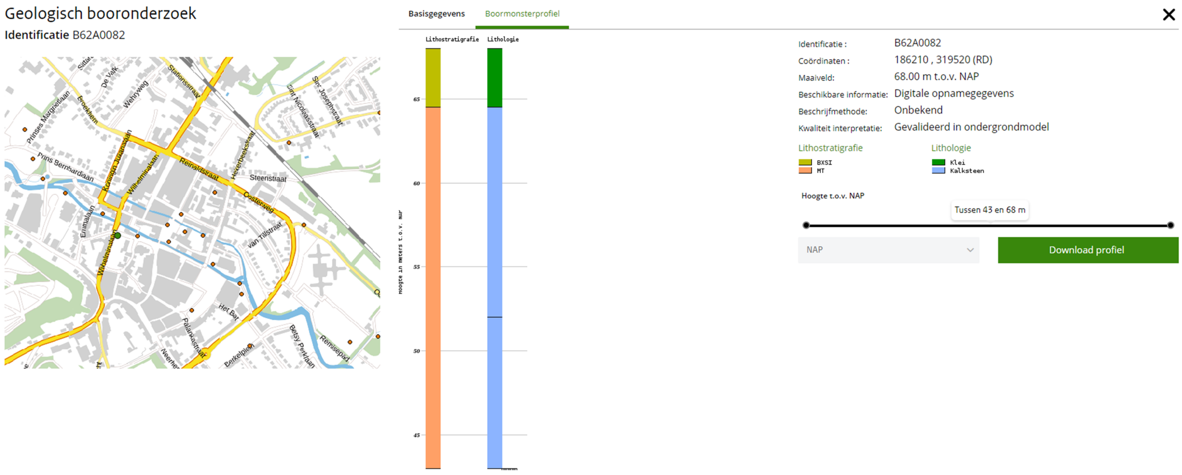


Figure B.6: Geological survey (TNO Geologische Dienst Nederland, 2022)

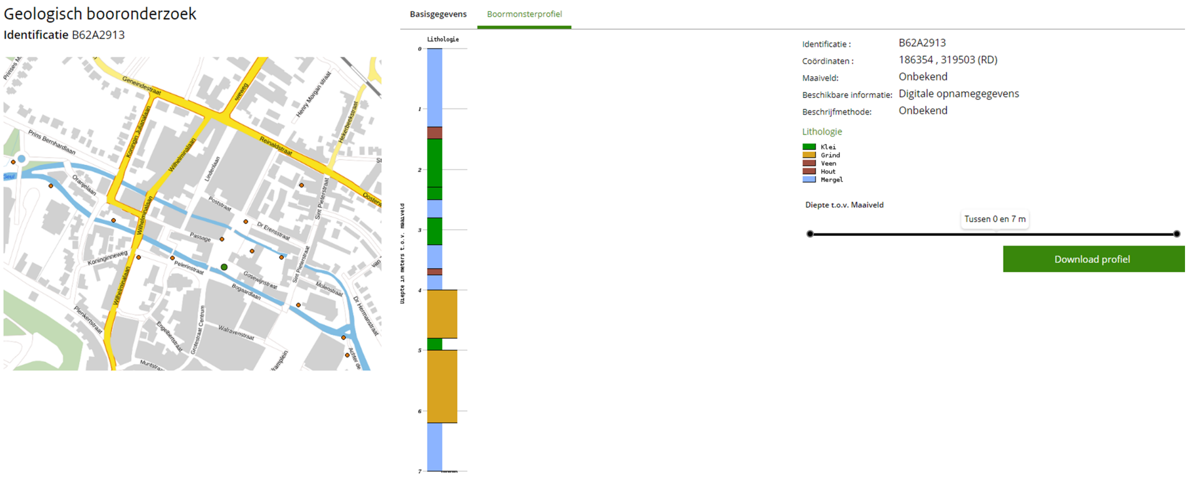


Figure B.7: Geological survey (TNO Geologische Dienst Nederland, 2022)

Figure B.8: Geological survey (TNO Geologische Dienst Nederland, 2022)

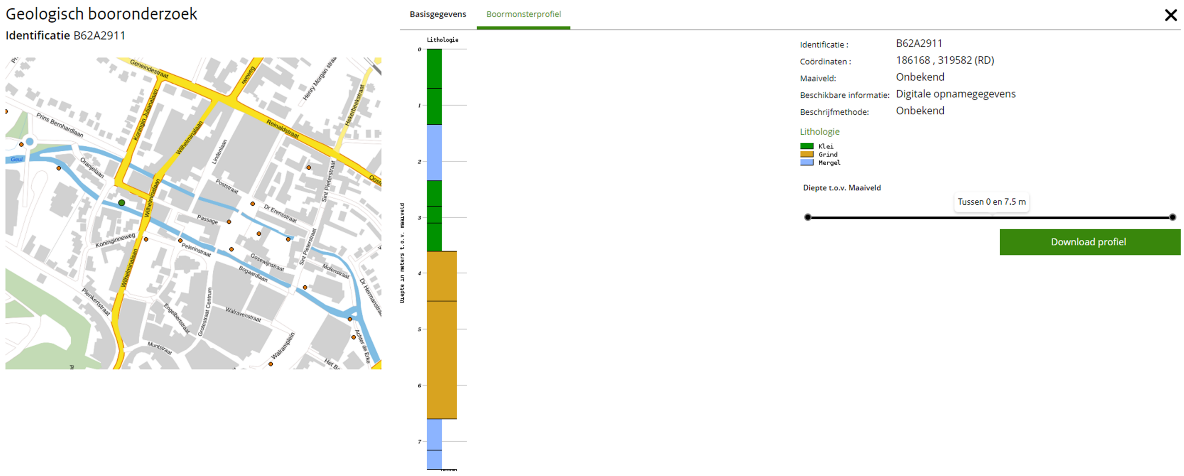


Figure B.9: Geological survey (TNO Geologische Dienst Nederland, 2022)

Figure B.10: Geological survey (TNO Geologische Dienst Nederland, 2022)

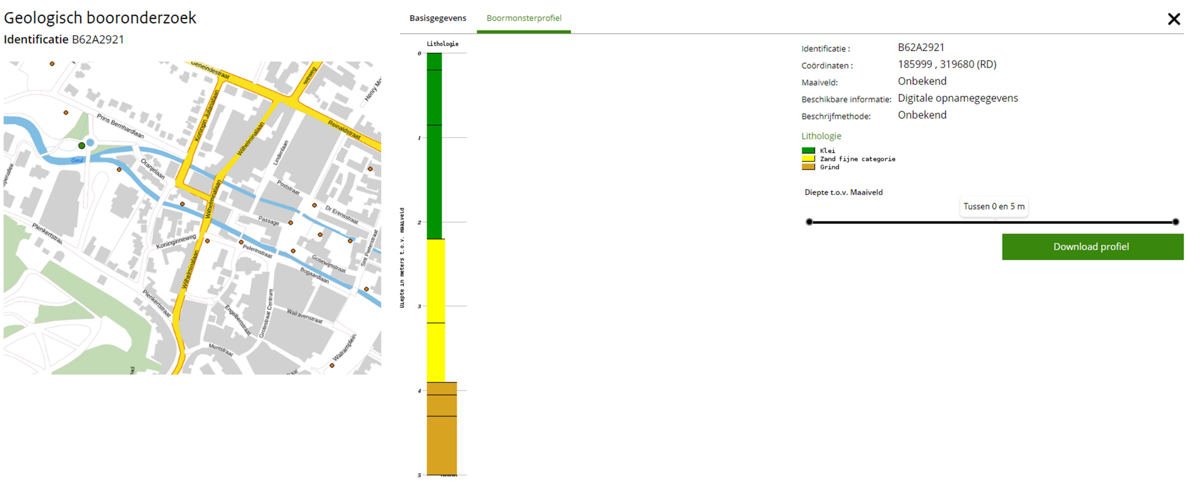


Figure B.11: Geological survey (TNO Geologische Dienst Nederland, 2022)

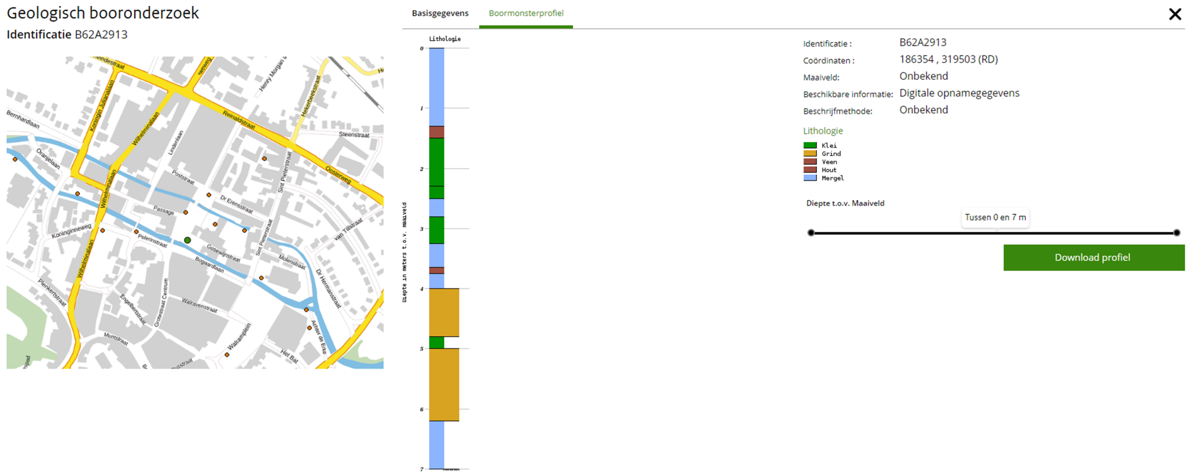


Figure B.12: Geological survey (TNO Geologische Dienst Nederland, 2022)

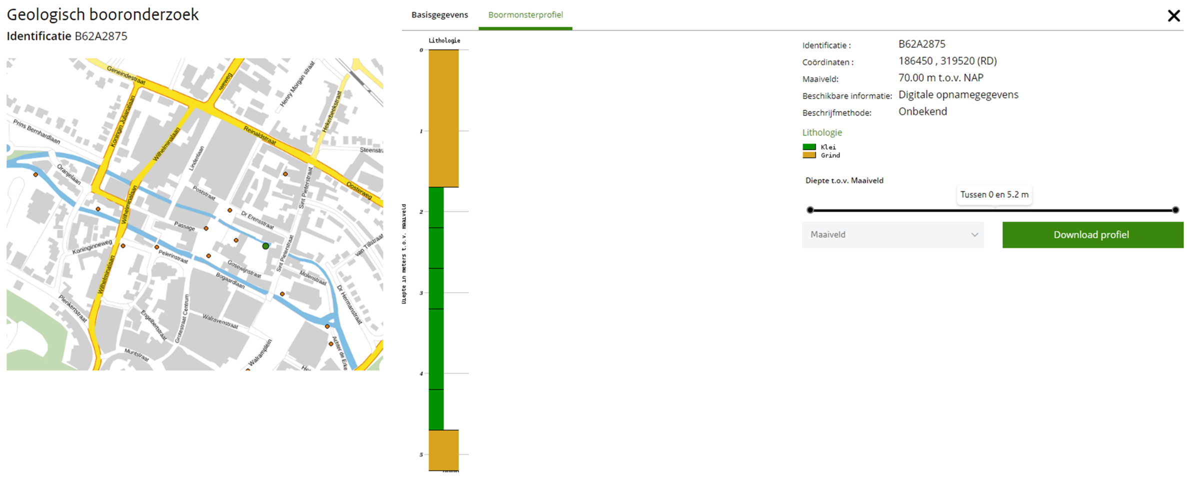


Figure B.13: Geological survey (TNO Geologische Dienst Nederland, 2022)

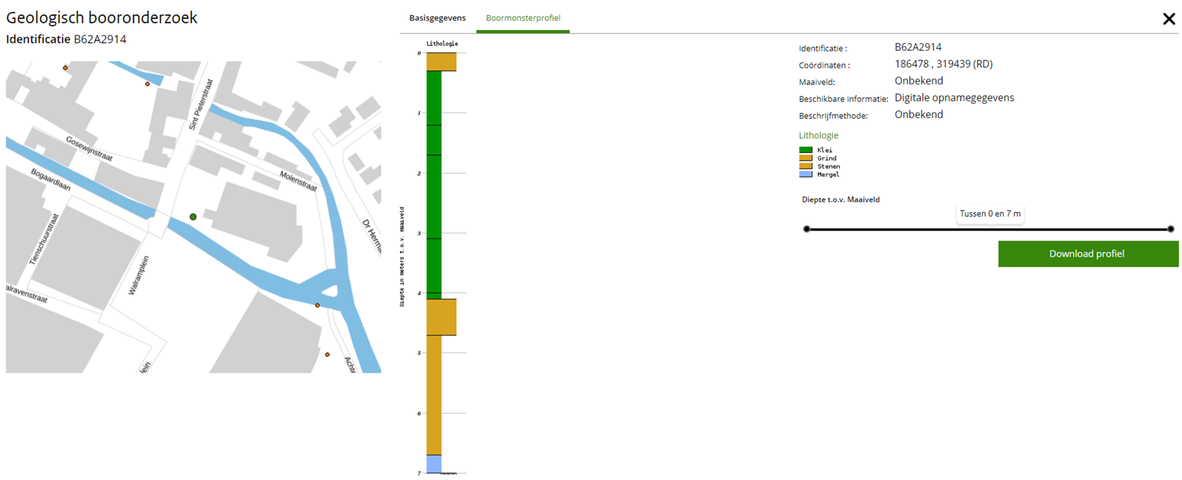


Figure B.14: Geological survey (TNO Geologische Dienst Nederland, 2022)

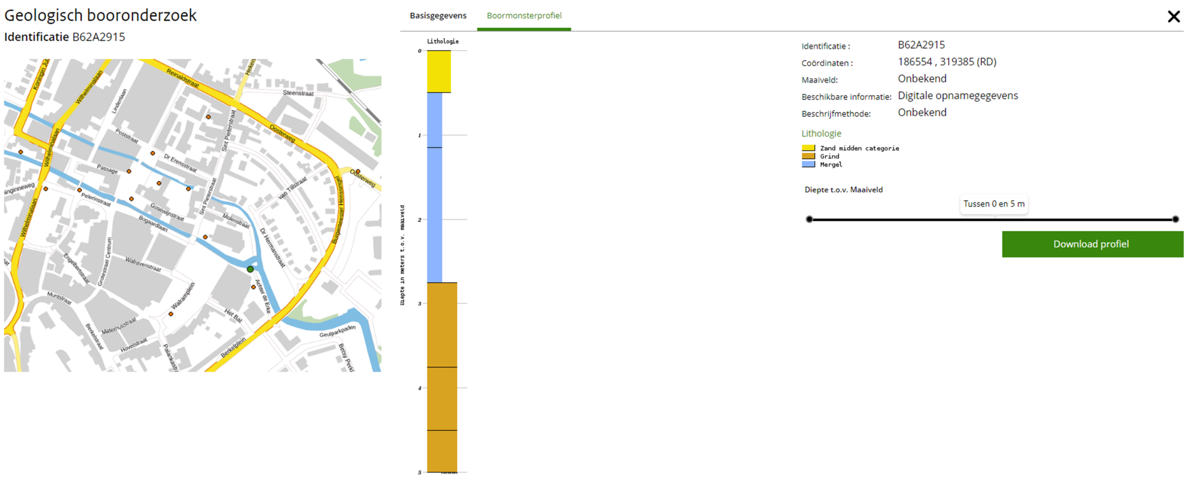


Figure B.15: Geological survey (TNO Geologische Dienst Nederland, 2022)

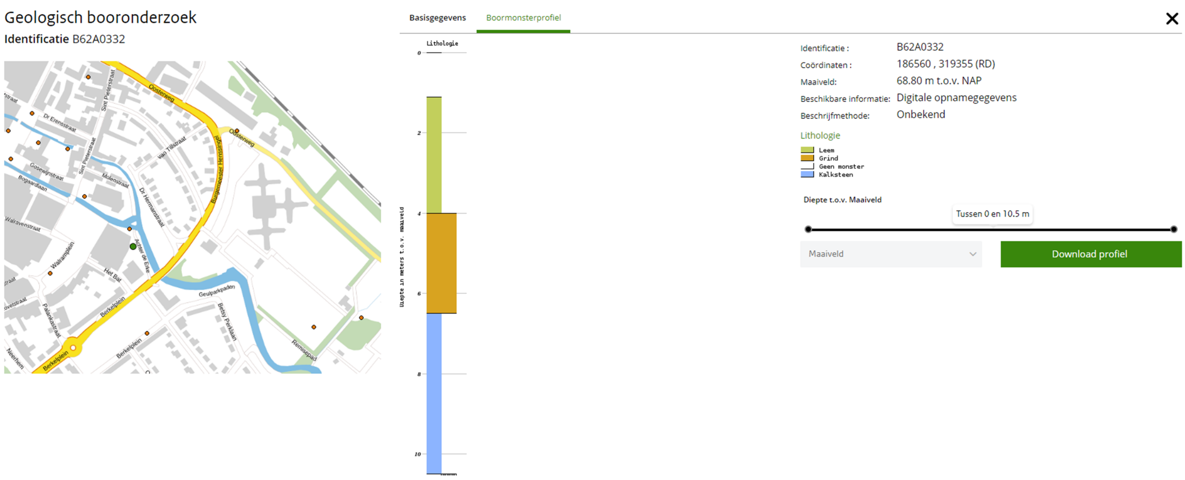


Figure B.16: Geological survey (TNO Geologische Dienst Nederland, 2022)

C
Basis of Design

This appendix provides extra information on the Basis of Design of Chapter 3. In Section C.1 the status quo is supported by an analysis of the location of the in- and outlet and the tunnel depth. In section C.2 the boundary conditions are supported by an analysis of climate change and bored tunnels.

C.1. Status quo

This section consists of an analysis of the location of the outlet and the tunnel depth.

Location outlet

This section consists of an analysis of the location of the outlet.

Outlet located at Opus park

The inlet and outlet 1 are determined by De Leeuw & Mondeel (2021). The location of both the inlet as well as the outlet are located at a public parking spot to minimise disruption to the public. From Figure 1.5 it follows that for outlet 1 approximately a 2 m difference in head is applied. This tunnel will be 800 m.

Outlet located at Brewery

Leijser & Nijhof (2022) had the idea of locating the outlet inside the newly constructed flood side channel which will be closed to the Geul to ensure that the water will not flow back into the city from this side. The capacity of the flood side channel is sufficient for this. From Figure 1.5 it follows that for outlet 2 approximately a 3 m difference in head is applied. This tunnel will be 1060 m.

(a) Connection of outlet 1 to the system

(b) Connection of outlet 2 to the system

Figure C.1: Connections of the outlets to the system

The outlet location is based on a combination of the proposed outlet location of De Leeuw & Mondeel (2021) and the outlet by Leijser & Nijhof (2022). The result is an outlet location where the tunnel smoothly rejoins the Geul, promoting straight streamlines and reducing the number of bends in the tunnel, while the building pit is still located at a parking site.

Tunnel depth

This section consists of an analysis of the tunnel depth.

3.5 m depth under the river

The depth of the tunnel will be just under the river, then the two tubes have to be on top of each other. This way there are no borings underneath houses. The boring follow the river.

10 m depth direct

The depth of the tunnel will be 10 m deep. The two tubes can be next to each other. This way the borings are far underneath the houses and will not interfere with other installations in the ground and foundations of houses.

For now, chosen to have to tunnel at 10 m depth to be sure not to interfere.

C.2. Boundary conditions

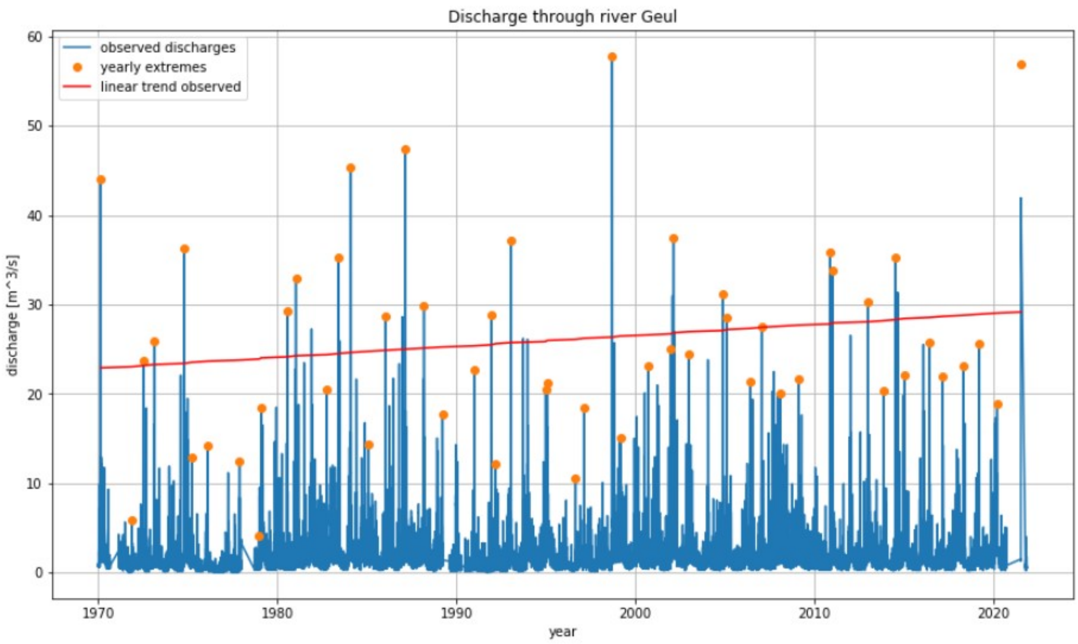
This section consists of an analysis of climate change and bored tunnels.

Climate change

This section consists of an analysis of climate change. On top of the 25-year return period in our current climate, climate change has to be taken into account to ensure a flexible design for the future. As no previous research has been done on this projection to the future regarding the Geul literature research has been done to project the following scenarios:

Linear projection of historic discharge data

Kallen et al. (2022) projected the historic mean maximum discharge linearly. To project this for 2122 the discharge is assumed to be 1.424 times higher than 2021. This means: 84 · 1.424=109 m^3/s. This minus the capacity of the tunnel should be 72.616 m^3/s, as the Geul is assumed to flood at 47 m^3/s.



this means that the expected mean maximum value of 2122 will be 1.4244487103417407 times higher than that of 2021

Figure C.2: Linear projection of historic mean maximum discharge data (Kallen et al., 2022)

KNMI projection of rainfall

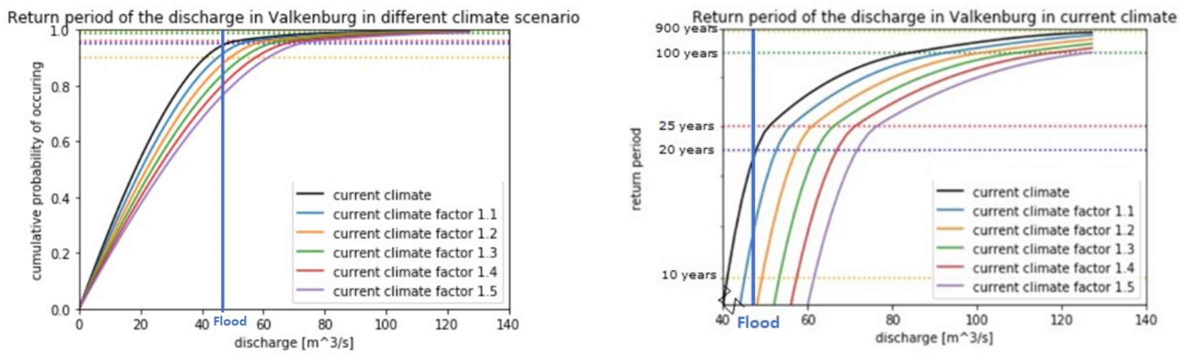
According to KNMI 48 h rainfall with a return period of 100 years in the climate of 2085 will come down to 114 to 145 mm compared to 2014's climate with 111 mm rain (van Heeringen et al., 2022). As the design will be made for 100 years (2122), the upper bound is used. Therefore a 30 % increase in rainfall is expected for such an event. When assuming the rainfall is directly correlated to the discharge the 30 % increase in discharge, therefore, 84*130%=109 m^3/s. This minus the capacity of 47 is 62.2 m^3/s KNMI (2021) supports this as the chance of heavy rainfall in Limburg is assumed to increase to 2040 with a factor of 1.2-1.4.

How the system reacts to this amount of rainfall is uncertain, due to the unknown amount of basins this does not necessarily translate to the amount of extra discharge, thus further research has to be done to find this out. As during the flood of 2021 in total 80 to 85 % of the rainwater falling in the Netherlands did not contribute to the flood and in Belgium, only 50 % of the rain fallen here was retained (Bureau Stroming, 2022). Therefore not all the rainfall contributes directly to the flood, however, the projections made are for 2085 and 2040 respectively, therefore the amount of discharge will increase significantly.

Sensitivity analysis based on the scenarios

The return periods with occurring discharges estimated by van Heeringen et al. (2022) are linearly multiplied with different factors to indicate the sensitivity of the system. These points with return periods are numerically interpolated with a piecewise cubic Hermite interpolating polynomial. This interpolation preserves monotonicity in the interpolation data and does not overshoot if the data is not smooth (Moler, 2004). This creates the smooth cumulative probability of occurrence graph see Figure C.3.

As can be seen from this sensitivity analysis is that the change in climate has an enormous impact on the discharge and return period. However, this is all still very uncertain or called deep uncertainty which means that "we do not know what we do not know" (Marchau et al., 2019).



(a) Cumulative probability of discharges occurring

(b) Return period discharges on a vertical logarithmic scale

Figure C.3: Sensitivity analysis of discharges with return periods (PCHIP) one-dimensional monotonic cubic interpolated

Drought situation

According to the KNMI dry springs and summers increase. The mean summer discharge decreased over the past few years from 3.07 m^3/s to 2.4 m^3/s after 1989 (Tsiokanos, 2022). With a return period of 100 years, the most extreme circumstance in the future climatic scenario is therefore considered to be near 0.

According to the KNMI, the Netherlands faces climate concerns in the next years due to an increase in drier springs and summers as well as more intense summer showers (KNMI, 2021) The patterns identified in this study are consistent with these climate scenarios. One of the key conclusions of this thesis is that the runoff patterns in the Geul are significantly influenced by climate variability. This makes it clear that the hydrological regime is predicted to be significantly impacted by projected changes in precipitation and temperature characteristics as a result of climate change. (Tsiokanos, 2022).

Bored tunnel

This section consists of an analysis of bored tunnels. The advantage of a bored tunnel is that it only takes up space above ground at the sites of the beginning and end shafts, while the groundwater level does not have to be lowered. It is important to take into account the settlement of the ground near the tunnel and it is important to realise that the measures to prevent calamities during the work at great depths are expensive and time-consuming (Molenaar & Voorendt, 2019).

Drilled tunnel

Initially drilled tunnels were considered less suitable for Dutch soil conditions as drilled tunnel requires a specific subsoil (Molenaar & Voorendt, 2019). In Valkenburg, the subsoil is made up of marl, flint and sand. This is suitable for drilling tunnels. Only coarse gravel would cause issues (Van Dijk, 2022). The boring of a tunnel using a drilling method is very expensive due to the drilling equipment needed and the mentioned measures needed to prevent calamities. The estimated cost for the proposed tunnel by Yvo would be 112-207 €, see Table C.1.

Pipe jacking

Pipe jacking involves forcing the tunnel element forward in the form of rings. This method is often used for pipelines and cable ducts (Molenaar & Voorendt, 2019). As this method is used often there is a lot of experience in this method and it requires less expensive equipment. This makes this technique cost-efficient. Due to the forcing of the tunnel element only smaller diameters are possible for this technique. The maximal inside diameter would be 3.5 m. Therefore multiple tubes are needed for Valkenburg. This would still be more cost-efficient compared to the boring method. The diameter of 3.5 m would the most optimum one cost-wise as the building cost per tunnel remains similar for different diameters while the discharge reduces much for a smaller diameter, see Table C.1.

Table C.1: Comparing Van Dijk (2022)'s drilling tunnel with De Leeuw & Mondeel (2021)'s pipe jacking tunnel

	Drilled tunnel	2 pipe jacked tunnels
Estimated costs [x million]	112-207 €	29 €
Diameter of the tunnel [m]	8	3
Length [m]	720	800
Discharge [m^3/s]	87	~40

D Reference projects

This appendix gives an overview of projects that have already been implemented, which could be useful for this project. The reference flood bypass tunnels are discussed in Section D.1 and the reference flood side channels are discussed in Section D.2.

D.1. Flood bypass tunnels

There are many different flood bypass tunnels applied in the world. In this paragraph, the flood bypass tunnel system is explained followed by several reference projects. There are of course many more examples than this.

General

The main function of a flood bypass tunnel is to rapidly convey floodwaters around the constrained area. The flood bypass tunnel occurs in densely populated areas and has a short length due to its cost and is most of the time constructed of stone, concrete, or metal. Compared to other bypass solutions they do not retain water and need the lowest amount of land as other structures can be located or built on top (Serra Llobet et al., 2021).

The challenges of the bypass are the bifurcation that originates after the construction of the flood bypass tunnel can potentially block inlets, reducing or eliminating the flow into the bypass or the main channel due to sediment and/or debris. On top of that, when the flood bypass tunnel reaches the end of its design life, reparation and replacement are complicated and expensive, due to the structures on top. The flood bypass tunnel takes the pressure off the main channel, which allows the main channel to retain or develop meanders, bars, bank vegetation etc. which contributes to habitat complexity but increases hydraulic roughness (Serra Llobet et al., 2021). However, this will not be the case for the Geul as the river is a canal in the city centre of Valkenburg. Flood bypass tunnels have typically a high flow velocity during floods, therefore the channels can be hazardous for humans, fish and other organisms (Serra Llobet et al., 2021).

Flood bypass tunnel Petra, Jordan

One of the first flood bypass tunnels is located in Petra, see Figure D.1. It dates back to the first century. Across the entrance to the city, known as the Siq, a dam was built across the Siq entrance to divert floods northward. The water diverted into a 100-meter-long channel that led into an 82-m long tunnel, both carved in bedrock. The tunnel was around 4.8 m wide and 7.5–8 m high (Serra Llobet et al., 2021).

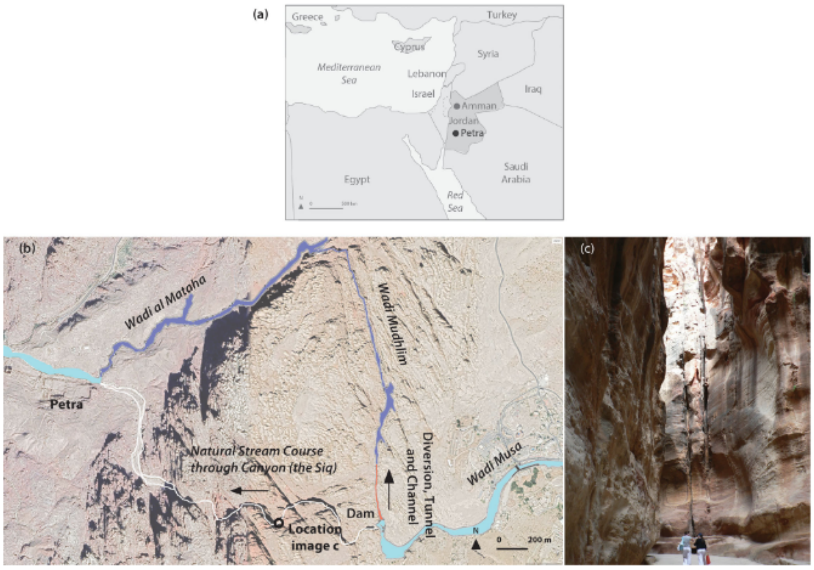


Figure D.1: Flood bypass tunnel Petra (Serra Llobet et al., 2021)

Flood bypass tunnel Geneva, Switzerland

A more recent example of a flood bypass tunnel is located at the River Aire in Geneva. In the 1960s, the river Aire was put in a straight underground culvert to make space for industries and a highway over the former course of the river (Département de l'Intérieur, 2003). The culvert capacity of 65 cubic meters per second was insufficient, and to prevent the industrial area from flooding, a flood bypass tunnel was completed in 1987. The flood bypass tunnel diverts 50 cubic meters per second directly to the Rhône, see figure D.2. In the 1930s, the river was partly canalised for agriculture, while downstream the river kept a more natural course. To protect this region as well the bypass is located upstream of this, to escape channelisation for flood control. Therefore the bypass to the Rhône is located at the transition from the channelised to remaining natural reach in the Aire (Serra Llobet et al., 2021), see Figure D.2.

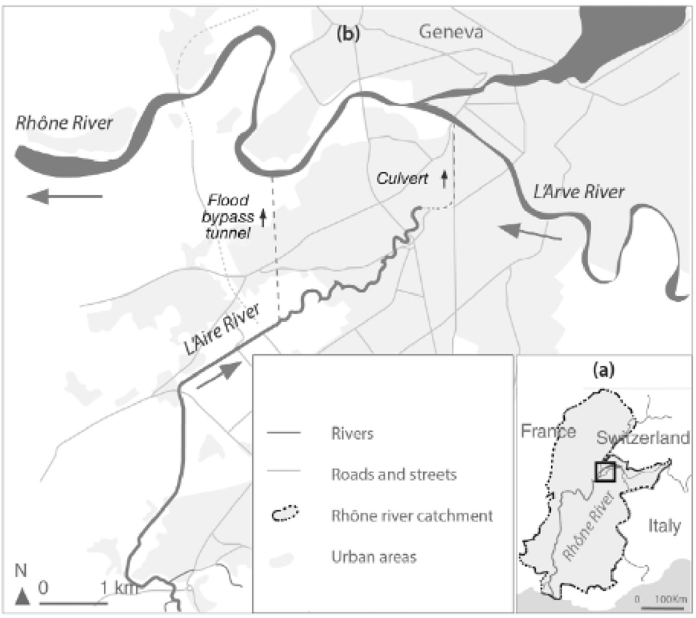
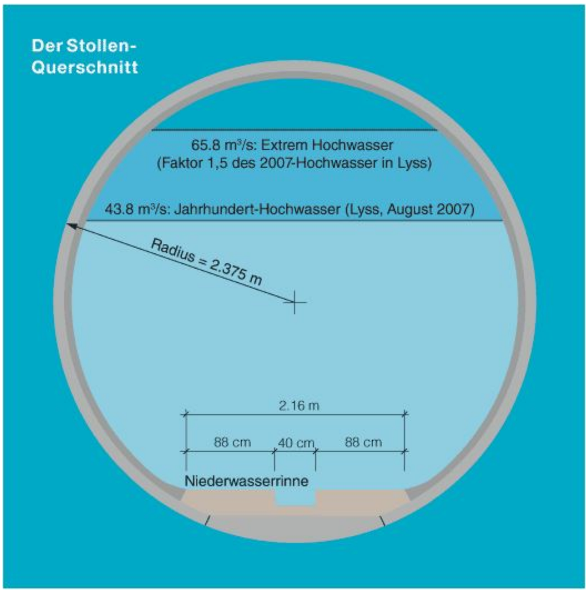


Figure D.2: Flood bypass tunnel Geneva (Serra Llobet et al., 2021)

Flood bypass tunnel Lyss, Switzerland

An even more recently built flood bypass tunnel is the one in Lyss. Due to the insufficient drainage capacity of the river Lyssbach, the village of Lyss flooded regularly. In the summer of 2007, it flooded three times. The damage from the floods in 2007 is estimated at around 100 million Swiss francs. To counteract this the discharge volume of the Lyssbach had to reduce drastically in an event of a flood. Various studies of technical feasibility and safety led to the decision of a flood bypass tunnel (Gemeinde Lyssbach, 2021). The flood bypass tunnel has a length of 2,570 m and a gradient of 0.58% over practically its entire length. The diameter of the tunnel is 4 meters. The capacity of the tunnel is around 70 cubic meters per second (Gemeinde Lyssbach, 2021), see Figure D.3a and D.3b.



(a) Cross-section tunnel tube of the flood bypass tunnel Lyss (Wasserbauverband Lyssbach, 2022)

(b) Overview flood bypass tunnel Lyss (Wasserbauverband Lyssbach, 2022)

Figure D.3: Flood bypass tunnel Lyss

The flood bypass tunnel has been in operation for almost 10 years and has saved Lyss several times from flooding. The total prevented damage has already exceeded the construction several times. The cost of the flood bypass tunnel of around 48 million Swiss francs. This includes the prevention of the flooding of Lyss during the storms of June 28, 2021. The amount of runoff would have caused damage of more than CHF 40 million in Lyss (Gemeinde Lyssbach, 2021).

The operation of the flood bypass tunnel is explained using Figure D.4. At (1) the structure lets water in. When the water level at Bielbrücke reaches a critical level (24 m^3/s) the structure will let the water accumulate behind the screen, rises and flow into the pond at (2). In the pond, the current is reduced and sediment can settle to prevent it from entering the flood bypass tunnel. At (3) a coarse rake makes sure that debris will not enter the flood bypass tunnel. After that, the water will go into the flood bypass tunnel at the inlet structure (4). The flood is discharged via a vertical shaft. The outlet structure will reduce the energy before re-entering the Lyssbach channel (Gemeinde Lyssbach, 2021).

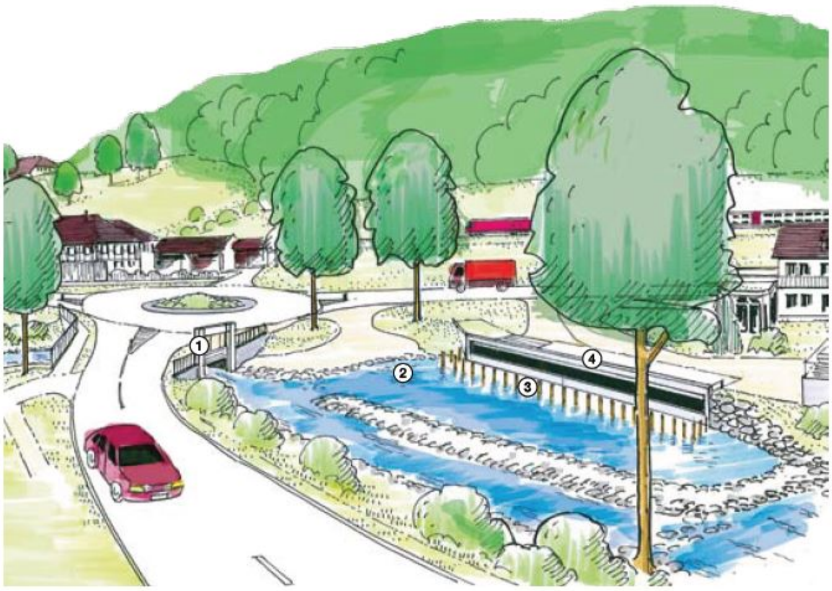


Figure D.4: Inlet of the flood bypass tunnel Lyss (Wasserbauverband Lyssbach, 2022)

Flood bypass tunnel Sarneraa, Switzerland

The flood catastrophe of August 2005 and other flood events made it clear that investments in flood safety in the Sarneraa Valley must be made. Therefore the Sarneraa Alpnach project was developed. The project consists of two parts. The Sarneraa Alpnach I project widens the channel to create space and integrate nature, while the Sarneraa Alpnach II project consists of the construction of a flood bypass tunnel. The expected total costs of these hydraulic engineering projects, both the flood bypass tunnel (Sarneraa Alpnach II) and Sarneraa Alpnach I are around CHF 200 million (Hochwasserschutz, 2022).

This section is focused on the flood bypass tunnel part of this project, however, the two projects intertwine well and could be considered in future research. The flood bypass tunnel is the central element of the Sarneraa flood safety project. It leads from the Sarnersee to just below the Wichelsee, see Figure D.5.

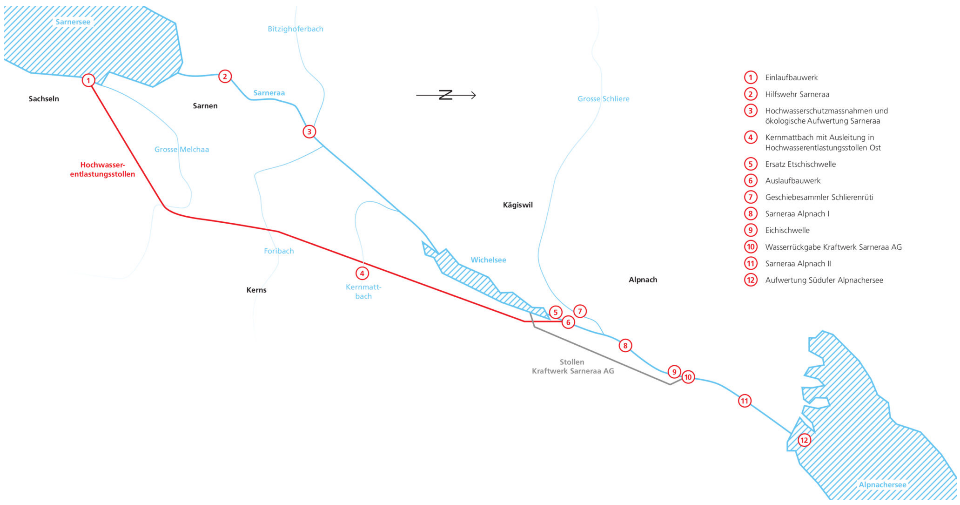


Figure D.5: Overview flood bypass tunnel Sarneraa (Hochwasserschutz, 2022)

In February 2018, construction work for the flood bypass tunnel started. The outflow capacity from Lake Sarnen will be significantly increased with the east flood bypass tunnel. The flood bypass tunnel leads from Lake Sarnen to below Lake Wichel and is around 6.5 km long. At high tide, it absorbs up to

100,000 litres of lake water per second from Lake Sarnen and drains it away without causing damage. With the help of the weir system in the outlet structure of the flood bypass tunnel and an auxiliary weir in the Sarneraa, above the Rütistrasse, the water discharge from Lake Sarnen will be regulated in future (Hochwasserschutz, 2022).

With a construction period up to 2024, the flood bypass tunnel with inlet and outlet structure is the largest component of the flood protection project. The tunnel runs along the right side of the Sarneraa valley from the Seehof area in Sachseln (inlet structure) to just below the Wichelsee dam in Alpnach (outlet structure). There it flows into the Sarneraa (Hochwasserschutz, 2022).

The flood bypass tunnel is designed as a pressurised tunnel. This means that the gates at the inlet structure are always open and the tunnel is constantly filled with water, see Figure D.6. If there is a risk of flooding, the gates in the outlet structure are opened in accordance with the regulation. When the gates are fully open, the discharge in the tunnel will reach the maximum capacity of 100 m³/s. Due to this large discharge capacity, the lake level would still rise to a maximum of 471.40 m above sea level in the event of an extreme flood like that of 2005. In comparison: In 2005, the maximum level of Lake Sarnen was 472.42 m above sea level.

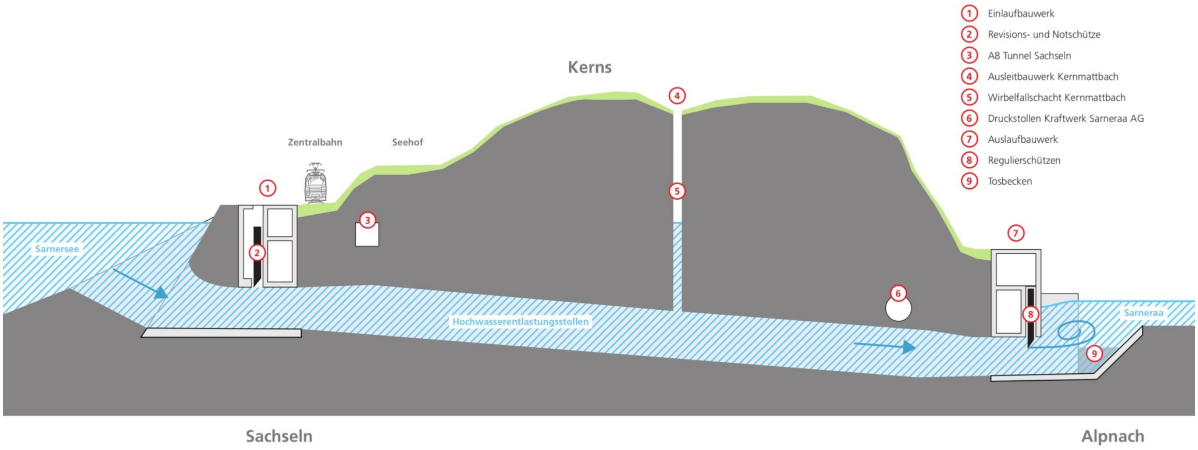


Figure D.6: Cross-section flood bypass tunnel Sarneraa (Hochwasserschutz, 2022)

The inlet structure forms the entrance to the flood bypass tunnel, see Figure D.7. In the event of an imminent flood, lake water can be drained through the tunnel and the lake level can be regulated in this way. In the event of an imminent flood, the inlet structure can hold up to a maximum of 100,000 litres of lake water per second and feed it to the Sarneraa via a flood bypass tunnel below the Wichelsee in Alpnach. The bottom of the inlet funnel is around 11 meters below the water level. As a result, most of the structure is constantly underwater and cannot be seen from the shore. To prevent driftwood from clogging the inlet to the tunnel, the inlet structure includes an upstream floating beam equipped with baffles (Hochwasserschutz, 2022).

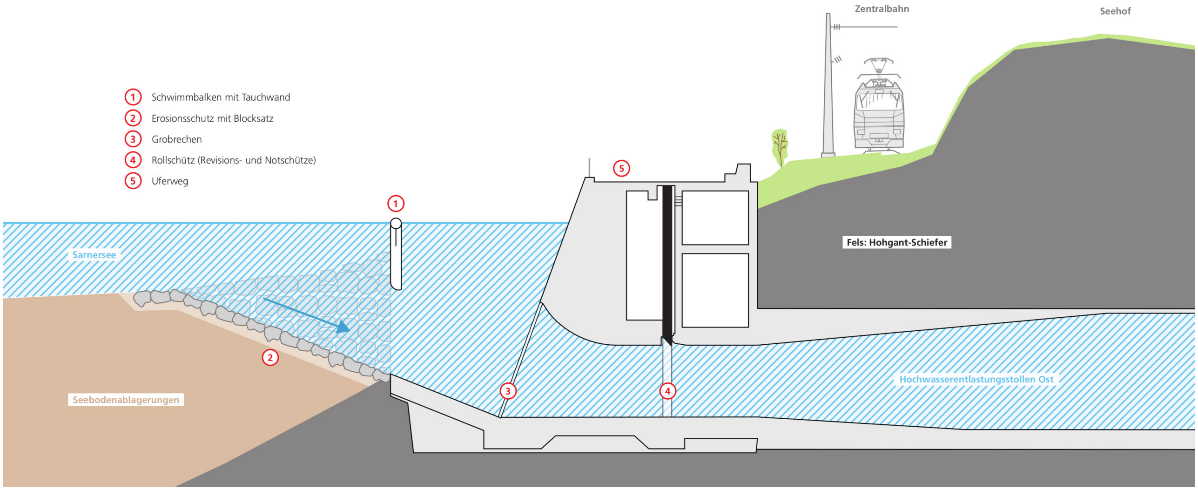


Figure D.7: Cross-section detail inlet flood bypass tunnel Sarneraa (Hochwasserschutz, 2022)

Flood bypass tunnel Thun, Switzerland

As a result of the floods of 1999 and 2005 at Lake Thun and bordering areas. The flood bypass tunnel was planned and built between 2006 and 2012. The tunnel was drilled with a boring machine from the outlet side. The flood bypass tunnel has an inner diameter of 5.5 meters, an outside diameter of 6.28 meters and is 1,129 meters long and it runs up to 15 meters below the ground level, see Figure D.8. The capacity is 110 to 115 m^3/s and the flood bypass tunnel has a storage volume of approximately 28,000 m^3. For the maintenance of the flood bypass tunnel, the water in the flood bypass tunnel can be drained through the permanently installed pump.

Thanks to a sophisticated system for early automatic detection, flood hazards can be avoided with the help of more water flowing out of Lake Thun at an early stage. This means that a retention volume for the increasing inflows can be created in Lake Thun. The flood bypass tunnel, which is always filled with water except for maintenance work, is opened and closed at the outlet structure. Normally, the flood bypass tunnel is closed.

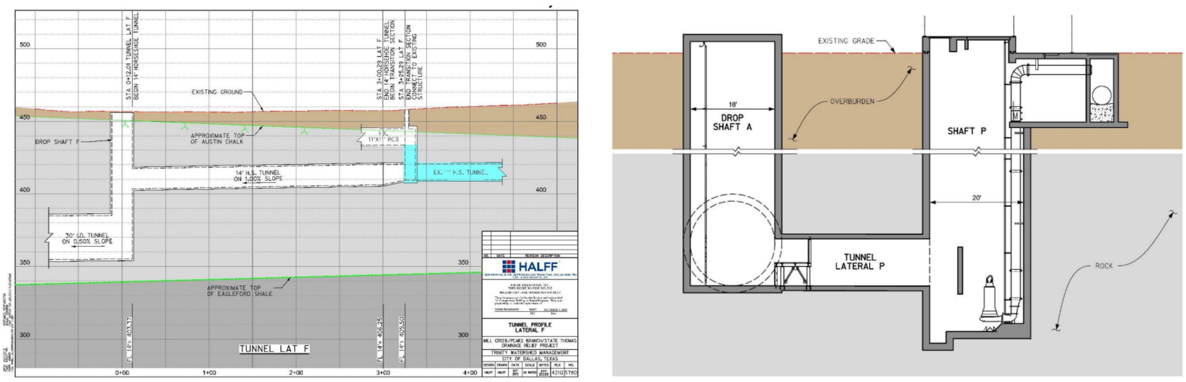
As of March 2009, the flood bypass tunnel was tested, and the impact on the residents was evaluated further down the Aare. The investments amounted to CHF 62 million. The flooding of 1999 and 2005 caused 170 million Swiss francs in damage at Lake Thun and bordering areas. The damage would have been half as expensive if the flood bypass tunnel had already existed at the time (Autor & Bruderer, 2020).

Figure D.8: Flood bypass tunnel Thun (Autor & Bruderer, 2020)

Flood bypass tunnel Dallas, Texas

According to City of Dallas (2022) the flood bypass tunnel of Dallas will reduce the probability of a flood to once every 100 years. The previous flood system, constructed 50 to 70 years ago, only provided two to five-year flood protection. By improving flood water management, the flood bypass tunnel will safeguard streets, residences, schools, and medical institutions. The flood bypass tunnel will have six inlet sites along the alignment, see Figure D.9. These inlets are drop shafts, see Figure D.10a. The dewatering happens by pumps, see Figure D.10b. The flood bypass tunnel has a length of 8.1 kilometres, a diameter of 9.1 meters to 10.7 meters and a discharge capacity of 424 m^3/s (Kallen et al., 2022). The project is scheduled to be completed in 2023.

Figure D.9: Overview flood bypass tunnel Dallas (City of Dallas, 2022)



(a) Flood bypass tunnel inlet Dallas (City of Dallas, 2022) **(b)** Flood bypass tunnel pumping station Dallas (City of Dallas, 2022)

Figure D.10: Flood bypass tunnel Dallas

Flood bypass tunnel San Antonio, Texas

The flood bypass tunnel of the San Antonio River is approximately 4,900 metres in length and has segmented liners made of precast concrete with an interior diameter of 7.3 meters (San Antonio River Authority, 2022), see Figure D.11. The estimated discharge capacity is 189 m^3/s (Kallen et al., 2022).

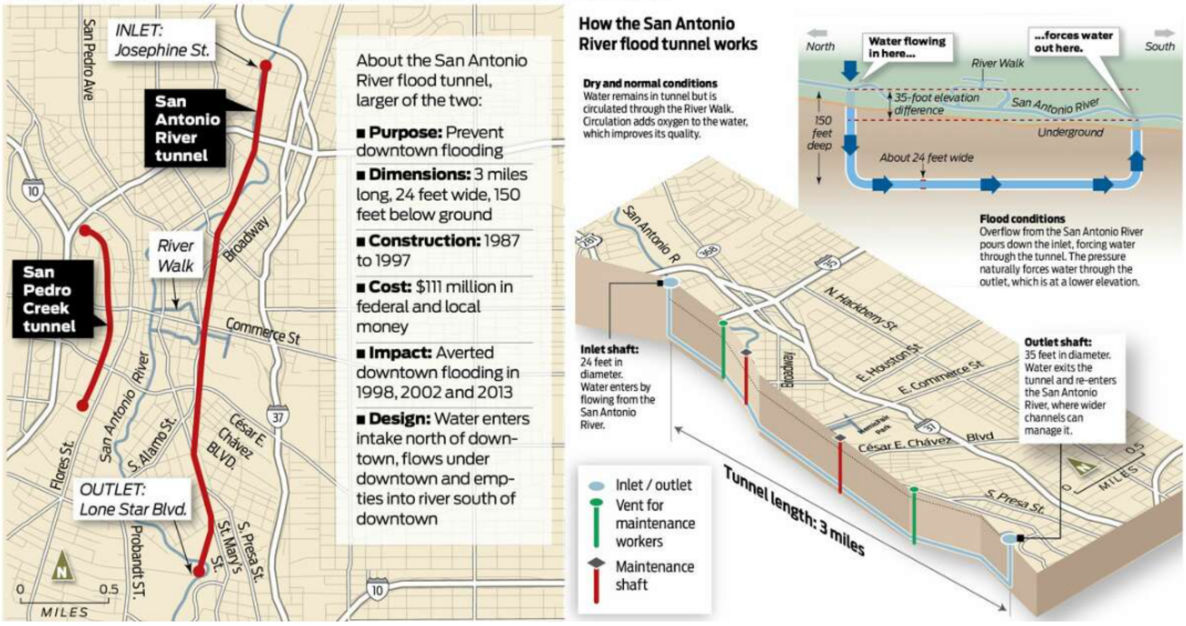


Figure D.11: Overview of the flood bypass tunnel of San Antonio (San Antonio River Authority, 2022)

In September 1993, the inlet structure was constructed, which includes an Ogee weir and an apron structure with a cast-in-place roof and trash racks, to direct water from the San Antonio River into the flood bypass tunnel, see Figure D.12. A sloped apron collects water behind the Ogee weir before the water flows into a vertical shaft that leads to the flood bypass tunnel. The flood bypass tunnel is disinfected and re-circulated by pumps to enhance the river's water quality. Through ventilation shafts, water can expel excess air, or "burp." To maintain oxygen levels, the flood bypass tunnel is filled with water to capacity and it is circulated continuously. At both the inlet and outlet, water is pumped and drawn back into the river (San Antonio River Authority, 2022).

Figure D.12: Inlet (Ogee weir) of the flood bypass tunnel San Antonio (San Antonio River Authority, 2022)

In May 1995, the outlet structure was constructed. It discharges from the flood bypass tunnel shaft to the San Antonio River. The outlet consists of trash racks, a roof and a stilling basin of cantilevered retaining walls. At the base slab of the outflow, the diameter of the tunnel shaft increases from 7 to 10 meters (San Antonio River Authority, 2022). A plaza, park facilities, channelisation, and a gatehouse for the flood bypass tunnel's water re-circulation are also part of this project. The total cost of the project is $111,400,000 (San Antonio River Authority, 2022).

D.2. Flood side channel the Netherlands

There are many flood side channels applied in the Netherlands. In this paragraph, the flood side channel is explained followed by two reference projects. The way the flood water flows into the flood side channels could be an example of how the flood water flows into the flood bypass tunnel.

General

Flood side channels are secondary channels that are connected to the main channel of a river, but are in general much smaller and convey much less discharge than the main channel. Side channels increase the discharge capacity of the river, and hence, reduce flood water levels.

Flood side channel Groene rivier, Roermond

The channel is also known as the Green River and was constructed in the early 1990s, see Figure D.13. The green river is designed to protect the centre of Roermond during high water in the Maas. This river, with a length of about 500 meters, forms a closed connection between the river Roer and the Maas under normal circumstances. At high water in the Meuse, the Roer is closed at the mouth and upstream to prevent the Meuse water counter-current from flooding the centre of Roermond. The Roerwater is then discharged via this green bypass. The projects costed f 15 million (Cobouw, 1996).

During the summer flood of 2021, the water discharge in the Roer rose to 300 cubic meters per second, whereas previous records hovered around 120 cubic meters. At the same time, the water of the Roer (and the Hambeek, a little further on) was also 'pushed back' by the large amount of water that flowed to Roermond via the Maas. This situation was discussed during the high water on 16 and 17 July, but the Water Board decided not to activate the system after all. If that had happened, there would have been a real risk of a dyke breach along the Groene Rivier, according to the Water Board, which could have flooded the city centre(van Well, 2021).

Figure D.13: Flood side channel Roermond (van Well, 2021)

Flood side channel Spiegelwaal, Nijmegen

Until a few years ago, Nijmegen was still a major bottleneck for floodwater safety. The river was wedged between the city and the moraine on one bank and the dike on the other. As part of Room for the River, an impressive solution was created in which nature and safety go hand in hand: the Spiegelwaal, an enormous flood side channel between the island of Veur-Lent and the old village of Lent(Ruimte voor de levende rivieren, 2022). The flood side channel made space for nature, see Figure D.14a. Sand martins and little plovers breed in the area and rare plants such as branched horsetail and little rock thyme grow. Even the rare dragonfly river rump has been seen. It also serves recreational purposes such as hiking, biking, fishing, rowing and running (Ruimte voor de levende rivieren, 2022).

(a) Flood side channel overview Spiegelwaal (Ruimte voor de levende rivieren, 2022)

(b) Flood side channel weir Spiegelwaal (Ruimte voor de levende rivieren, 2022)

Figure D.14: Flood side channel Spiegelwaal

The flood side channel reduces the water level to a maximum of around 34 cm, see Figure D.15 The total land area of the project was 250 hectares. For the construction of the flood side channel, 50 homes/businesses were demolished. Subsequently, more than 5.2 million cubic meters of soil were moved. This created a flood side channel of 4 km long, 200 meters wide and 8 meters deep in relation to the floodplain ground level, and 14 meters in relation to the quay and dyke heights, see Figure D.14b. The new flood defence is 1.2 km long. A seepage screen was placed in the flood defence, 1.6 km long, 20 meters deep, and 80 cm wide. An extra wall for reinforcement was built around the pillars of the Railway Bridge, 23 meters deep and 1.5 meters wide. The total construction time was 36 months and the output remained within the budget of 358 million euros. The bypass ensures that the water in the Waal drops 34 cm if the Spiegelwaal starts flowing with the river during extremely high water (Spiegelwaal Nijmegen, 2020), see Figure D.15.

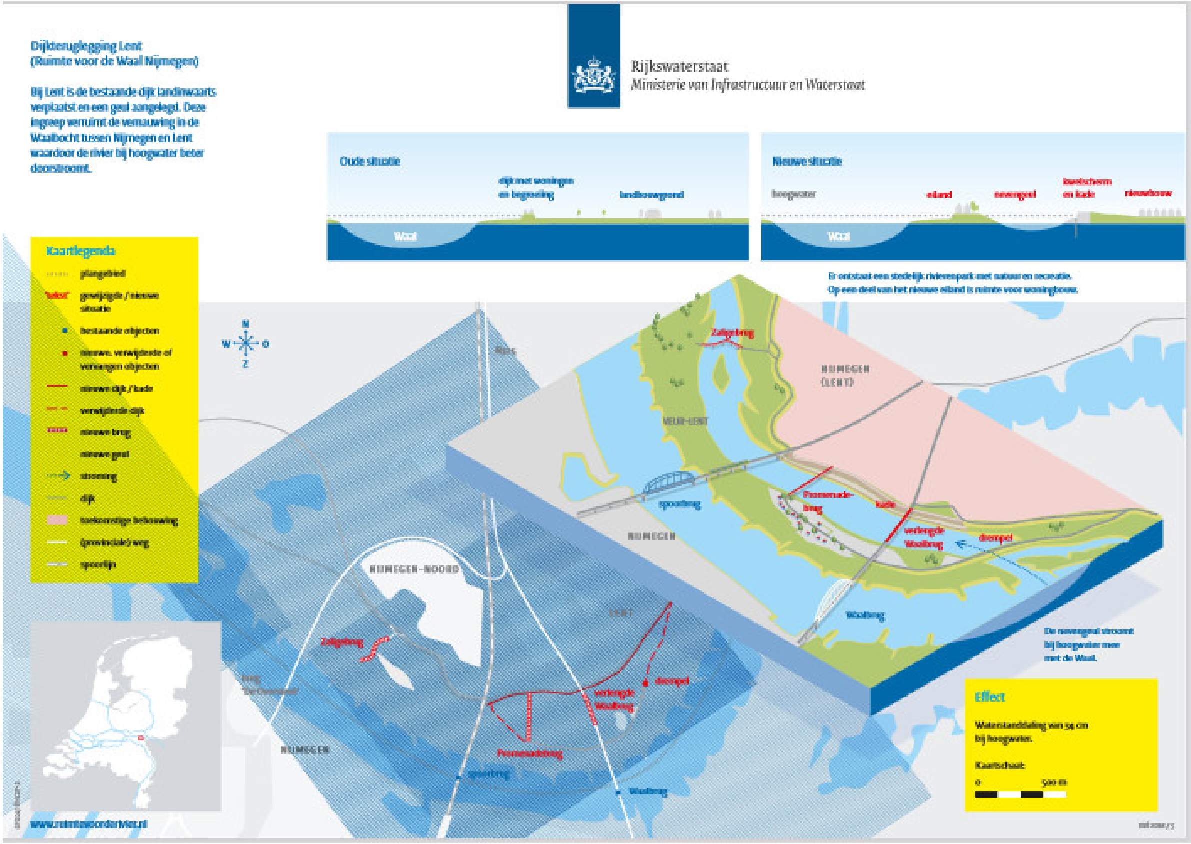


Figure D.15: Flood side channel Spiegelwaal(Spiegelwaal Nijmegen, 2020)

E

Hydraulic design

This appendix provides extra information for the hydraulic design of Chapter 5. In Section E.1 the coefficients of the primary hydraulic design are illustrated. Section E.2 gives background information about the air transport in a descending tube.

E.1. Coefficients primary hydraulic design

Figure E.1 depicts the trash rack head loss. Note that in the figure the headloss is divided by feet and in feet. Therefore the coefficients are the same in meters.

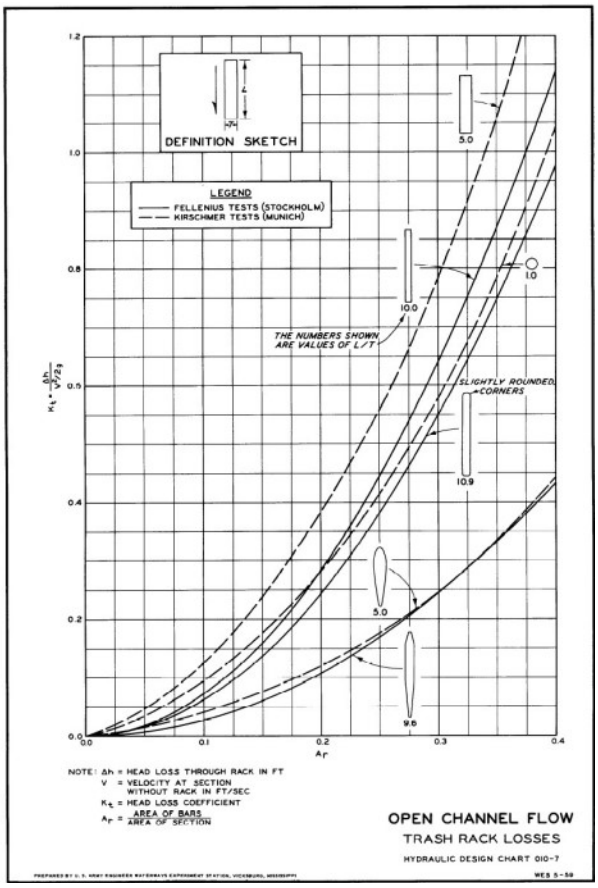


Figure E.1: Coefficient trash rack (Army Engineer Waterways Experiment Station, 1977)

Figure E.2 illustrates the coefficients of pressurised flow.

Xi; inflow
0,4 - 0,5
0,8 - 1,0
0,2 - 0,3
0.10
0.00
0,5 + 0,3 cos α + 0,2 cos^2 α
Xi; outflow = 1

Xi;bends	Degrees						
R/D	90	60	45	30	15	10	5
1	0.30	0.26	0.21	0.15	0.08	0.05	0.03
2	0.16	0.14	0.11	0.08	0.04	0.03	0.01
3	0.12	0.10	0.08	0.06	0.03	0.02	0.01
4	0.11	0.10	0.08	0.06	0.03	0.02	0.01
5	0.09	0.08	0.06	0.05	0.02	0.02	0.01
6	0.09	0.08	0.06	0.05	0.02	0.02	0.01
7	0.08	0.07	0.06	0.04	0.02	0.01	0.01
Elbow ○	1.20	0.55	0.28	0.15	0.05	0.04	0.02
Elbow □	1.40	0.64	0.33	0.18	0.06	0.05	0.02
*Two bends near each other result in a double Xi							

Xi = n (A2/A1 -1)^2	
Degrees	n
6.00	0.14
10.00	0.20
15.00	0.30
20.00	0.40
30.00	0.70
40.00	0.90
50.00	1.00
60.00	1.10
70.00	1.10
90.00	1.10

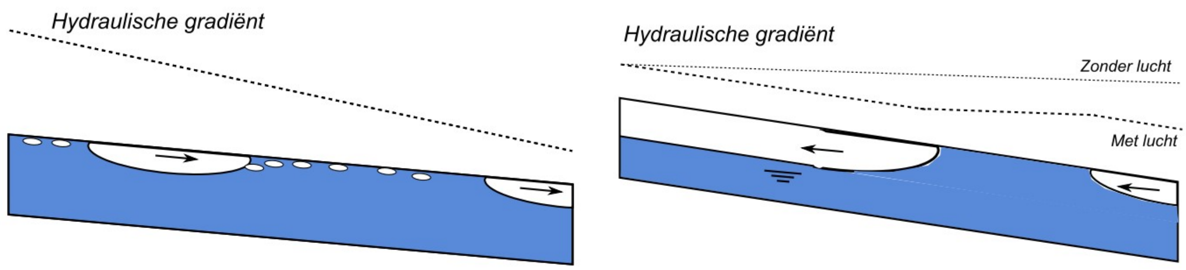
Wall roughness (k) in m (Tubes)				
Material	Norm	Min		Max
Stone/Brickwork	0.002	0.0007	-	0.01
Cast iron	0.001	0.0002	-	0.002
Concrete	0.001	7E-05	-	0.02
Wood	0.0006	0.0007	-	0.002
Steel	0.0005	3E-05	-	0.002
HPE /PVC	0.0005	5E-05	-	0.001
GVK/AX	0.0005	0.0001	-	0.001
Glas	-	3E-05	-	8E-05
Grass	-	0.001	-	0.005
Ground	-	0.008	-	0.7
Stone/Loose	-	0.01	-	0.4
Roughness depends on age and way of manufacturing/production.				

Figure E.2: Coefficients pressurised flow (Army Engineer Waterways Experiment Station, 1977)

E.2. Air transport in descending tube

A descending tube transports air less efficiently, which means the air bubble can lead to significant head losses. The stagnation of air bubbles occurs when the tube lays steeper than the hydraulic gradient. The head losses can be between 1 and 5 m per 1000 m with an angle of just 0.1 to 0.3 degrees. If the hydraulic gradient is steeper than the slope of the tube the air bubbles will be transported, however, if the velocity of the air bubble is less than the velocity of the water, see Figure E.3b. Due to the balance of forces, the air bubble will transport to a lower-pressure area. If the hydraulic gradient is less steep than the slope of the tube the air transport is dependent on the flow rate in the tube, see Figure E.3b (Tukker et al., 2012).

The transport capacity of air bubbles in descending pipes of sinkers and boreholes is very small. It takes hours to break down a gas bubble through flow. So it is possible that the tube can fill with gas over the entire descending length over time. The behaviour of air bubbles in pressurised pipes is unpredictable and strongly depends on the angle of the descending tube. If air bubbles stagnate and accumulate in the pipeline, this can lead to considerable local energy losses.



(a) Behaviour of the air bubbles if the descending slope is less steep than the hydraulic gradient (Tukker et al., 2012)

(b) Behaviour of the air bubbles if the descending slope is steeper than the hydraulic gradient (Tukker et al., 2012)

Figure E.3: Behaviour of the air bubbles in a descending slope

Due to this phenomenon, the ideal profile of the tube without a chance of air entrapment would be a flat or ascending profile. As stagnation only occurs in descending pipe sections. If a descending pipeline is necessary, steep angles of inclination are preferred (> 60°, with 90° slopes achieving the greatest gas transport). Research shows that gas can be discharged more quickly at steep angles of inclination.

From research by Lubbers (2007), the required flow number for gas volume transport in a vertical pipeline (angle of inclination is 90°) is 0.4 and is therefore significantly smaller than the required flow number at 60°, see Figure E.4. Thus, gas bubbles are most efficiently descended in a vertical pipe.

Figure E.4 is divided into 3 areas. In area 1 little or no gas transport takes place. The flow rate in the pipeline is too low to transport gas bubbles to the lowest point, and gas is mainly discharged by dissolution in the liquid resulting in low gas disposal. As a result, the entire descending pipe section can be filled with gas, which gives energy loss equal to the height difference over the descending pipeline.

In area 2 the volume of gas is broken up into several separate gas bubbles that spread along the length of the descending pipe. Small bubbles are carried along by the liquid flow and transported past the lowest point. The energy loss with a continuous gas supply becomes significantly smaller (the largest reduction in energy loss occurs in this area) as the flow number increases.

In area 3 all gas volumes are discharged, regardless of size. The energy loss caused by the gas volumes is negligible (depending on the amount of gas supplied).

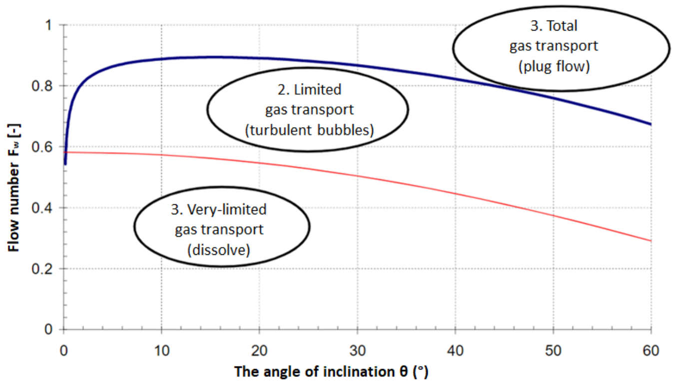


Figure E.4: Air in tube *translated* (Tukker et al., 2012)

F

Multi criteria Analysis

In this appendix, the factors of the criteria are weighted in Section F.1 and the design alternatives are evaluated in Section F.2.

F.1. Weighting factors of the criteria

The weighting factors of the criteria are determined in discussion with the municipality (Gemeente Valkenburg) and waterboard (Waterschap Limburg) (Gemeente Valkenburg aan de Geul, personal communications, November 28, 2022) & (Waterschap Limburg, personal communications, December 6, 2022). Each criterion is given a score of 1 to 5 based on the importance of the criterion. Less important criterion scores 1 and very important criterion scores. 5.

1. **Effectiveness - 3**;
   With the assumption that the design discharge is met the effectiveness of the design alternative is neutral important. Therefore the effectiveness criterion scores 3 out of 5.
2. **Reliability - 4**;
   The reliability of the design alternative is important, therefore this criterion is scored 4 out of 5.
3. **Control - 2**;
   The need for control of the design alternative is less important to the municipality, as long as the design alternative functions, therefore this criterion is scored 2 out of 5.
4. **Safety - 5**;
   Safety is very important to the municipality, therefore this criterion is scored 5 out of 5.
5. **Maintainability - 4**;
   Maintainability is important to the municipality, therefore this criterion is scored 4 out of 5.
6. **Effect on the environment - 2**;
   The effect on the environment by integrating the environment is not as important to the municipality as functionality, therefore this criterion is scored 2 out of 5.

Additionally, the municipality mentioned that the track of the tunnel could be lengthened in the future to protect care homes upstream and perhaps other development projects. This will be further elaborated on in the discussion.

F.2. Evaluation of the design alternatives by scores

The evaluation of the design alternatives is also done by scores. The evaluation is determined in a discussion the waterboard (Waterschap Limburg, personal communications, December 6, 2022). Each criterion is given a score of 1 to 5 based on the importance of the criterion. Less important criterion scores 1 and very important criterion scores. 5.

Effectiveness

The effectiveness of the flood bypass tunnel is determined by the discharge capacity of the flood bypass tunnel during design operation. A design that has a higher discharge capacity during the design operation receives a higher score for this particular criterion.

Design Alternative 1: Passive flood bypass tunnel - 4

The discharge capacity of Design Alternative 1 is 55 m^3/s. As the flood bypass tunnel is empty before a flood, a buffer is created. This adds time to the system, during which other communities can be warned. The effectiveness of the first design alternative is therefore given a 4 out of 5 as the design discharge is met, which is slightly less than the second design alternative, however, the buffer increases the effectiveness.

Design Alternative 2: Active flood bypass tunnel - 3

The discharge capacity of Design Alternative 2 is 58 m^3/s. This is slightly more than the first design alternative, however as the flood bypass tunnel is normally filled. Therefore sludge could settle, which decreases the diameter of the tunnel which decreases the discharge capacity. The effectiveness of the second design alternative is therefore given a 3 out of 5.

Reliability

The reliability is determined by the number of steps that have to be taken during the operation, which makes the system more complex. As more steps have a large chance of failure. A design that has a higher reliability receives a higher score for this particular criterion.

Design Alternative 1: Passive flood bypass tunnel - 4

This design alternative is reliable as the structure is passive without moving parts. Therefore not a lot of steps need to be taken during the operation. When the water level is reached the system will be activated by itself. Someone from the waterboard has to go there to monitor the situation. The non-movable design alternative has as a pro that it will be dry before and first will be filled, which adds some extra time to the system in which other communities can be warned (Waterschap Limburg, personal communications, December 6, 2022). As the system is relatively simple the chance of failure is small, therefore the score is high, 4 out of 5.

Design Alternative 2: Active flood bypass tunnel - 2

This design alternative is less reliable as the structure is active and requires moving parts. When a flood is predicted the gates have to be opened. The mechanism operates electrically therefore in case of a power outage failure is very likely, as happened last time. Also, someone has to go there to monitor the situation when opening and make sure it works (Waterschap Limburg, personal communications, December 6, 2022). This acquires multiple steps, which makes the design alternative more complex. Using a failure tree of the steps, the risk of failure is increasing with multiple steps. Therefore the score is low, 2 out of 5.

Control

The system's control is determined by the degree of regulation over the discharge that flows through the flood bypass tunnel during operation. A design that has a degree of regulation over the discharge receives a higher score for this particular criterion.

Design Alternative 1: Passive flood bypass tunnel - 3

This design alternative has no movable parts, therefore, there is no degree of regulation over the structure, however, due to the weir the water level is controlled by the structure. Therefore the value is neutral, 3 out of 5.

Design Alternative 2: Active flood bypass tunnel - 4

This design alternative has movable gates therefore there is a degree of regulating the structure. Generally, the gate will be open or closed, but opening the gate halfway to control the amount of discharge could be possible as well as opening the gates before high water when a flood is predicted. Therefore the value is 4 out of 5.

Safety

Safety is determined by the limitation of access of civilians to the flood bypass tunnel. A design that has a danger of falling in the flood bypass tunnel lowers the score for this particular criterion.

Design Alternative 1: Passive flood bypass tunnel - 2

The safety of this design alternative for the limited access of civilians and preventing the danger of falling in, especially for children and dogs etc, is guaranteed by placing fences along the inlet of the tunnel, next to other required measures to ensure safety, however as the Ogee weir is completely open, the design alternative scores 2 out of 5.

Design Alternative 2: Active flood bypass tunnel - 3

The safety of this design alternative for the limited access of civilians and preventing the danger of falling in, especially for children and dogs etc, is guaranteed by completely covering and closing off the flood bypass tunnel. Therefore this design alternative scores higher than the first design alternative, 3 out of 5.

Maintainability

Maintenance ensures that the flood bypass tunnel is operational during a flood. Therefore the structure must be easy to maintain with good access for possible inspections and possible replacements. A structure that is easier to maintain receives a higher score for this particular criterion.

Design Alternative 1: Passive flood bypass tunnel - 4

Design Alternative 1 is easier to maintain with good access for possible inspections. The tunnel has to be pumped out after every use, which removes sludge and which makes maintenance easy. Due to the simple system, the maintainability is high, thus this scores a 4 out of 5.

Design Alternative 2: Active flood bypass tunnel - 3

Design Alternative 2 has good access to possible inspections and possible replacements. However, the gates have to operate at least once a year to make sure that the system functions. Preferably, the objects are tested twice a year (spring and autumn) and after use, they need to be checked for damages (Waterschap Limburg, personal communications, December 6, 2022). With the testing of the gates, the system is flushed and will take some sludge with it. However the settled sludge will make it more difficult to maintain, so pumping out when maintaining, and removing the sludge is also necessary, to prevent forming of a layer of clay can at the bottom of the tunnel. As this decreases the diameter of the tunnel which decreases the discharge. Due to the more complex system, but the good access, the maintainability is neutral, thus this scores a 3 out of 5.

Effect on the environment

The effect on the environment is determined by the structure's integration into its surroundings, and the impact the structure has on the environment in terms of nature-friendliness. Therefore, the score for this particular criterion is lower for a stand-alone structure that is not incorporated into its surroundings and is not environmentally friendly.

Design Alternative 1: Passive flood bypass tunnel - 3

Design Alternative 1 is relatively integrated into the environment by the implementation of the structure in the surroundings. Except for the weir and the fences along the inlet- and the outlet and a co-current channel, there is nothing visible of the structure, which makes it neutral. The effect the structure has on the environment in terms of nature friendliness is that fish can enter the tunnel, but afterwards will be pumped out, which makes it neutral 3 out of 5.

Design Alternative 2: Active flood bypass tunnel - 2

Design Alternative 2 is barely integrated into the environment by the implementation of the structure in the surroundings. The gates are clearly visible as well as other components mentioned for Design Alternative 1, which makes it negatively impacted. The effect the structure has on the environment in terms of nature friendliness is that fish can enter the tunnel, but afterwards will be stuck until the gates are tested again and the water is refreshed, which makes it negatively impacted 2 out of 5.

G

Cost Analysis

In this Appendix, the cost analysis is elaborated. The construction costs are discussed in Section G.1 and the maintenance costs are discussed in Section G.2.

G.1. Construction costs

The pricing per unit for specific operations or materials is based on information retrieved from Witteveen + Bos. The construction costs consist of the jacking costs, shaft costs, bed protection, and depending on the type of design alternative on the gates/control house and/or the measures downstream. An estimation of unforeseen, general, profit and risk and client costs follows this. The overview is given in Table 6.3.

Jacket

For the pipe jacking method, different costs are considered. For the mobilisation, demobilisation and drilling facilities which are needed a price of € 1.200.000 is estimated. Both design alternatives need this thus this is one unit for both. The pipes in between the two points are needed. They have a unit price of € 3.500 /m, thus both design alternatives 850 m pipe is needed twice. This also counts for the pipe jacking itself with a unit price of € 4.500/m.

Shaft

For the pipe jacking method, a shaft is needed which requires different costs. The shaft itself approximately 10m deep (10mx15m) has a unit price of € 2.000.000. As one is needed at the inlet and one at the outlet, two are needed. To build the passive inlet construction (inside the shaft) for Design Alternative 1 a unit price of € 1.000.000 is estimated due to the unique shape of the Ogee weir. For the inlet of Design Alternative 2, the same cost as the passive outlet is used as these are the same. For the construction of the passive outlet, a unit price of € 500.000 is estimated.

Gates and control house

For Design Alternative 2 on top of the passive in- and outlet design a gate and control house is needed to control the gates. The gates are estimated to have a unit price of € 250.000. This means there will be 4 gates needed two at both inlets and two at both outlets. The control house which includes MEP (mechanical, electric, plumbing) has a unit price of € 500.000, which is needed twice, once at both the in and outlet locations.

Bed protection

As bed protection is out of the scope of this thesis a rough estimate is taken to calculate the costs of bed protection. The length of the bed protection is estimated with a rule of thumb based on the depth of the water, see Equation G.1. In this case, 3.5-meter water depth results in bed protection with a length of 35 meters. The thickness is assumed to be a meter, and of the width, the inlet and outlet width is used. For Design Alternative 1 this could be roughly 20 m for the inlet and 10 m for the outlet. This results in 1050 m^3. For Design Alternative 2 this would be roughly 10 m for both the in and outlet. This results in 700 m^3. The bed protection is estimated to have a unit price of 50 €/m^3.

$$L_{bedprotection} = 10 * h_0 = 10 * 3,5 = 35m \tag{G.1}$$

where:	$L_{bedprotection}$	[m]	=	Estimated length of the bed protection
	h_0	[m]	=	Estimated water depth

Downstream

For Design Alternative 1, there has to be a co-current channel constructed 900 m long. The co-current prevents the streaming back of the water into the tunnel and increases the discharge capacity downstream and also in the flood bypass channel downstream. The co-current channel is estimated to have a price of € 2.000 / m. For Design Alternative 2, the channel only needs widening of the Geul to the flood bypass channel, and widening this channel also to increase the discharge capacity downstream. The widening of the channel is estimated to have a price of € 1.000 / m. To widen the Geul two bridges need to be replaced for both design alternatives which have an estimated price of € 1.000.000 per bridge.

Total construction costs

To account for unforeseen costs 20% of the total costs are calculated extra. To account for General Costs, Profit and Risk (AKWR) an 18% of this total is taken and for costs client (engineering/tender) 20% is taken on top of that.

G.2. Maintenance costs

The pricing per unit for specific operations or materials is based on information retrieved from Witteveen + Bos. The maintenance costs consist of the pumping out, general maintenance and depending on the type of design alternative on the replacement of gates, testing of the system, electrical & ICT and mechanical costs house and/or the measures downstream. An estimation of unforeseen costs follows this. The overview is given in Table 6.4.

Pumping out

For pumping out the tunnel a unit price of € 5.000 is estimated. For Design Alternative 1, this is estimated to happen every year. This is after usage to ensure water quality. As Design Alternative 2 has the tunnel filled and closed off the pumping out of the tunnel will likely happen once every 25 years.

General maintenance

For general maintenance of the tunnel, a unit price of € 500.000 is estimated. For both design alternatives, general maintenance is estimated to take place once every 25 years

Replacement of gates

For Design Alternative 2, the replacement of gates needs to take place. This is estimated to be once every 40 years for all 4 gates. For the replacement of gates, a unit price of € 200.000 is estimated.

Testing of the system

For Design Alternative 2, the testing of the system needs to take place. This is estimated to be twice a year in spring and autumn. The testing will be done by the waterboard after which possible repairs need to take place (Waterschap Limburg, personal communications, December 6, 2022). For testing of the system, a unit price of € 10.000 is estimated.

Electrical & ICT

For Design Alternative 2, the replacement and/or improvement of the electrical & ICT is estimated to take place once every 15 years based on the lifetime requirements of the basis of the design. A unit price of € 7.500 is estimated for this.

Mechanical

For Design Alternative 2, the replacement and/or improvement of the mechanics is estimated to take place once every 25 years based on the lifetime requirements of the basis of design. A unit price of € 50.000 is estimated for this.

Total maintenance costs

To account for unforeseen costs 20% of the total costs are calculated extra.

H

Add-on opportunities

In this appendix, add-on opportunities have been looked at. Using a sewage pipe as a flood bypass tunnel is discussed in Section H.1. Adding a barrier in the Geul upstream of a flood bypass tunnel is discussed in Section H.3 as well as creating a retention area upstream of a flood bypass tunnel is discussed in Section H.2.

H.1. Sewage pipe

Currently, a sewage pipe runs beneath the majority of the centre of Geul in Valkenburg (from the Walram weir to Polfermolen); this could be investigated further to serve as a flood bypass tunnel. The pipe is 2.50-meter-wide and 1.50-meter-tall and will likely need to be extended and detached from the rest of the sewer system. The additional discharge capacity of this pipe is roughly 5 m^3/s, although this is dependent on the inlet and outlet being optimised. Moreover, the tube now has a drainage function, which must be compensated for (van Heeringen et al., 2022).

As the design is intended for a flood once every 100 years, a flood bypass tunnel of at least 37 m^3/s is needed. The discharge of 5 m^3/s is small compared to the discharge needed. The remaining discharge of 32 m^3/s still has to be accounted for. On top of this another drainage pipe is needed to be constructed to compensate for this, therefore the problem is just transferred and therefore this option is not explored further in this thesis.

H.2. Retention area

Due to a retention area upstream, water can be buffered which could increase the water level from roughly 69 m to 73 m, thus creating a 4 m difference in head extra. This increases the discharge significantly. On top of this it creates a retention area of roughly 742,000 m^2 extra, see Figure H.1. Assuming the bathymetry is linear (optimistic assumption), the average depth of this area will be 2 m, this creates a buffer of 1,484,000 m^3 retention.

Figure H.1: Size of a proposed retention area

The retention area is used as an inundation area, see Figure B.3. Together with some extra area depending on the current elevation of the land, less construction is needed, see Figure H.2.

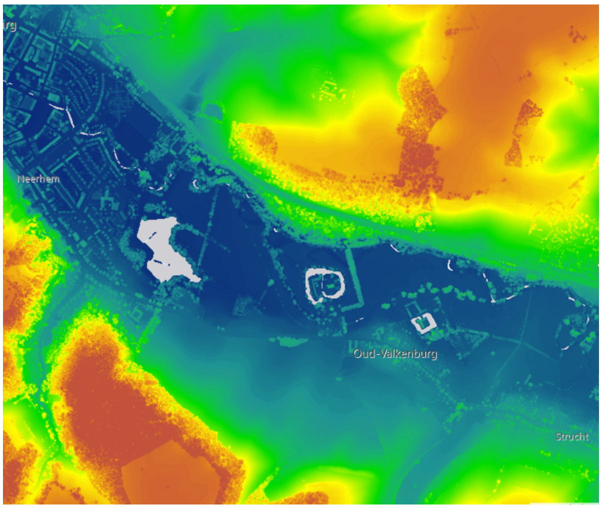


Figure H.2: Elevation difference in the area upstream

This design is multifunctional as the flood bypass tunnel is combined with a retention area upstream. The retention area will buffer the water, which will increase the head, which will increase the discharge in the tunnel while creating a retained area of water during the flood. This retention area when there is no flood will be continued to use as a recreation area. The retention area will contain wadi to aid water into the groundwater. It might be possible to retain water in drought as well which adds another function (more research is needed).

The design could be social and participial, as the retention area will continue to function as a recreation area when there is no flood while creating an education area as well. Extra/waterproof facilities will be applied too, for example, space for recreation outside, sport/play/paths facilities. This is decided by the residents. On the slopes of the dikes around the retention solar energy panels can be placed and by inundating the area frogs can lay their eggs in the puddles.

To apply this concept barriers are needed in the Geul next to the inlet of the tunnel. This gives another obstacle in the Geul. Also, construction in Natura 2000 is necessary, thus might consider a smaller retention area of only the Geulpark to prevent building in Natura 2000.

Figure H.3: Connection of the retention area with the rest of the system

H.3. Barriers in de Geul

Adding a barrier in the Geul together with a retention area upstream together could also guarantee a significant difference in head to prevent submergence of the Ogee weir. During the flood of 2021, the water naturally weir-ed due to the barriers in the Geul. Therefore here this option is neglected for now and could be researched later.

I

Sustainable opportunities

In this appendix sustainable opportunities have been looked at using the seven sustainable design principles of Witteveen + Bos (2022).

I.1. Seven sustainable principles

The seven sustainable principles are:

1. Building with nature; use natural processes. Not applicable to this project.
2. Circular design; use alternative materials. Construction should have minimal impact on the environment and minimize carbon emissions. Due to the current nitrogen crisis in the Netherlands and its certainty this part is neglected. However future research should take this into account.
3. Multifunctional; use a multifunctional design. Preservation ecological values and enhances the local biodiversity.
4. Social design; increase the social impact. Minimisation of the disruption for the public by not demolishing houses, but building in the public space. Also minimize noise, settlements, and traffic and ensure the safety of the citizens. The structure should integrate well with the surrounding.
5. Participate design; engage the environment in the design.
6. Trias; minimize the energy needed.
7. Flexible design; make the design adjustable for the future. The design should be designed for the future, thus climate change has to be taken into account. However, due to its uncertainty more research has to be done to apply this requirement.

I.2. Sustainable opportunities

Different reference projects which apply Witteveen + Bos (2022)'s seven sustainable principles have been looked at. Inspired by these projects the following ideas came up:

1. **Building with nature**;
   Not applicable to this project.
2. **Circular design**;
   Use alternative materials for example recycled concrete or concrete using biobased cement, both reduce carbon emissions.
3. **Multi-functional design**;
   Possibilities of multi-functional design are using the flood bypass tunnel in draught to retain water and/or using it as a diving cave. If pumped dry the tunnel can have multiple functions if a flood is not predicted. For example, holding an event. The tunnel can also be combined with a retention area upstream. The retention area will buffer the water, which will increase the head. This will increase the discharge in the tunnel while creating a retained area of water during the flood.

This retention area, when there is no flood, will be used as and recreation area. The retention area could also contain wadi to aid water into the groundwater (more research is needed on the groundwater in Valkenburg). Another function which can be added is enhancing the local biodiversity, by creating nesting space for the bats for example the inlet. Create space for the frogs by using the retention area to flood once every while. Also, limiting construction in the Natura 2000.

4. **Social design**;
   The tunnel design can be socially used to serve an educational purpose. The possible retention area upstream could function as a recreation area and education area as well. Here different facilities can be applied to attract social, for example, space for recreation outside, and sports/-play/paths facilities.
5. **Participative design**;
   The possible retention area design can be designed using participation, by letting the residents decide what kind of facilities they think are needed in the recreation area. Think of sport/play/paths facilities.
6. **Trias**;
   Minimizing the energy needed can be applied using no pumping in the system, also no moving elements could be a way of reducing energy. The use of alternative materials will prevent unnecessary waste. Another example could be to turbine the water in the flood bypass tunnel to gain energy, however, this will not be profitable as the tunnel will only function once every 5 years or so for a short time. Also, the discharge will not be sufficient enough. Which will then be an unnecessary waste. Another idea is that at the retention area dikes solar energy panels can be placed.
7. **Flexible design**;
   A possible flexible design is to take climate change into account for the lifespan of the project. Flexible design can also translate into a physically flexible design, which intakes extra discharge in the future. Therefore leaving options open to for example adding an extra tunnel by having extra inlet space.