Cost-effective levee design for cases along the Meuse river including uncertainties in hydraulic loads

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Cost-effective levee design for cases along the Meuse river including uncertainties in hydraulic loads

by

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Preface

This MSc Thesis reflects the final part of the Master of Science degree in Hydraulic Engineering at the Civil Engineering and Geosciences Faculty of the Delft University of Technology. The research is performed under guidance of the Delft University of Technology in cooperation with Royal HaskoningDHV and Waterschap Peel & Maasvallei.

I like to thank many people for their support and cooperation during my graduation thesis. In the first place I thank my direct supervisors: Timo Schweckendiek, Enno Kuipers and Bas van Lammeren for their helpful feedback, enthusiasm and guidance during the thesis. Many thanks to Prof. Matthijs Kok for his support and advice. My thanks to Jules Verlaan too, who helped me especially in the field of LCCA.

Furthermore I would like to thank my colleges at Royal HaskoningDHV, for all their advices and feedback regarding the contents as well as the process of the graduation thesis. It was great to experience the company in where I was able to talk to hydraulic-, management-, LCCA-, and engineering- specialists.

The MSc thesis topic originates from the Waterschap Peel & Maasvallei. Designing levees, operating with a limited budged and dealing with all kinds of stakeholders and preferences is not an easy task. In my view, looking for opportunities and considering innovative ideas characterize the water board. One of the largest struggles during my thesis was to find an appropriate scope which includes both; the large variety in aspects the water board (and engineering agencies) faces as well as the detailed level of some aspects to form comprehensive results.

Many colleges of the Water board gave me insight into their levee management difficulties, took effort to help me whenever I need and are very open for discussions. Therefore my thanks to these colleges as well. At last I would like to thank my friends, family and especially my fellow students for their encouragement, supported and help with the English language.

At last I like to give some thoughts to the lately published report giving answers to data which are assumed in this report. On the 16th of September 2014 the Delta-committee presented the Delta Programme 2015. This program gives insight into the new safety philosophy. The new annual flood probability of Arcen has to be less than 1/300. The program reported an annual flood probability of 1/1.000 for the city-centre boulevard of Venlo. within this research a safety level of 1/1.250 is used for both case studies. Some results, like the most economical attractive robustness of the levee height, are strongly related to these changes. The calculated robustness is overestimated, but this does not change the basic methodology. Furthermore the discount rate will be (re)investigated according to the report. The belief exist that the 5,5% total real discount rate can be to high due to economical and political reasons (DP2015).

B. Broers
Delft, December 2014
After the flood of 1993 and 1995 in the Netherlands, flood risks in Limburg became an important discussion. Each flood defence system along the river Meuse needs to be provided with a safety level with an annual failure probability of 1/250. In the nearby future a new safety philosophy will be introduced entailing new safety norms and calculation techniques. The water board needs to construct levees including the knowledge of this new safety philosophy. Additionally the water board faces more problems like limited space or an intense infrastructure around the current levees.

Uncertainties in hydraulic boundary conditions influence the levee design, too. An over- and underestimation of these conditions may lead to a cost-inefficient levee design. Moreover, innovative alternatives may be good alternatives. Including the uncertainties in the levee design alternative analysis can lead to a better decision process in where these innovations become more attractive.

The objective of this MSc-thesis is to investigate which balance between levee investments and robustness is most cost-effective during the levee design alternative analysis, specified for cases along the river Meuse in Limburg. Robustness within this balance is related to the uncertainties in future hydraulic boundary conditions. The main research question of the MSc-thesis therefore is:

What are cost-effective levee designs including uncertainties in hydraulic boundary conditions?

This research uses two case studies; Arcen and Venlo. The case study of Arcen is used to define, design and screen levee design alternatives. Furthermore an optimization procedure is developed with this case study. In Arcen some of the important problems are that the present levee passes through peoples back yards, it has a very high present failure probability and there is not much available space. The second case study will be used to confirm or reject the preliminary conclusion drawn in the Arcen case study. In Venlo, along the river Meuse a boulevard is present. This boulevard is partly protected by a demountable stop-log system which currently does not fulfil the present safety assessment. Future developments, preferences of other stakeholders and large service costs leading to a re-analysis of the levee design.

Levee design alternatives are formed by combining a levee alignment with a green or structural design type. Secondly design criteria are formulated to create sketch designs per levee alternative. Next, these alternatives are screened with a multi-criteria analysis based on criteria like costs, constructibility and hydraulic impact. Alternatives which turn out to be most attractive of the MCA are referred to as realistic alternatives. These will be investigated further with help of LCCA.

Life Cycle Cost Analysis or LCCA is a widely acknowledged technique to support the alternative analysis. One of the methods is the so called Net Present Value (NPV) calculations which include investments as well as benefits. This methods suits perfectly to investigate the robustness and the effect of hydraulic boundary condition uncertainties to the alternative analysis.

The investments in LCCA includes among others: Project-, maintenance-, service- and end-of-life costs within the analysis period of a project. The benefits of a levee design can be estimated by calculating the flood risk reduction between the present and new situation. The NPV of a levee design alternative can be found by subtracting the present values of the investments from the benefits. A discount rate and economical growth rate is used to perform these calculations.

The theoretical framework of LCCA as stated above is adapted to the optimization procedure. The aim of this procedure is to find the optimum levee height \( h_1 \) by seeking the robustness \( \Delta h_1 \) argument wherein the NPV-function attains its maximum value.
Here the robustness is the levee height above the required height which is necessary to fulfil the present safety standards (annual exceedance probability of 1/250). Overtopping and overflow is within this model the only failure mechanism.

Figure 1 illustrate the optimization procedure where the robustness ($\Delta h_1$) is varied. The hydraulic boundary condition uncertainties are included probabilistically (design water levels related with a 100 year analysis period $t=T$). If the robustness is large enough ($h_1 \geq h_2$) no major maintenance is necessary. Otherwise the levee height has to be increased ($\Delta h_2$) at $t = T_m$. The uncertainties in hydraulic boundary conditions are calculated with a Monte-Carlo simulation. The maximum physically possible discharge of $4.600 \, m^3/s$ of the river Meuse is included into the optimization procedure.

Within this research three different hydraulic boundary condition uncertainty aspects are analysed quantitatively, and assumed to be independent:

- The uncertainty in future river widening projects;
- the uncertainty in statistics;
- the uncertainty in forecasted climate changes.

The uncertainties of all hydraulic aspects are assessed with respect to the new safety levels. For both cases, an annual failure probability of $1/1.250$ is used.

Berkhof et al. (2013) selected areas which can be used as possible future river widening projects. Two scenarios are analysed for the river Meuse. A triangular distribution is chosen to represent the future river widening projects.

A normal distribution is assumed for the uncertainties in statistics as it resembles an error in the use of various distributions as well as the differences in discharge measure methods and the extrapolation error.

Rules of thumb are used to assume a uniform distribution for future water level increases in order to include uncertainties in forecasted climate change. A 95% confidential interval is chosen between the most positive and negative extreme scenario’s drawn up by the KNMI (van den Hurk et al. 2006).
A sensitivity analysis with help of a FORM-simulation shows that all three hydraulic boundary condition aspects contribute significantly.

The outcome of the optimization procedure of case study Arcen shows a large robustness of 0.6 meter for the case study Arcen for all levee design alternative. Furthermore the impact of including these hydraulic conditions to the alternative analysis is relatively equal. Project investments are strongly governing and the effect of the discount- and economical growth rate to the alternative analysis is limited.

The analysis has been validated by a second case study in Venlo. Note that no full validation can be concluded as it is only one extra case study. There is, in contrast to the preliminary conclusion drawn in case study Arcen, no full dominance of the project investments. Service costs and major maintenance costs are significant, too. An optimal robustness is found of 0.8 meter for all realistic alternatives above the present safety level of 1/250. The discount- and economical growth rate does influence the alternative analysis significantly, but mainly between the 0 and 2.5%. Again the relative difference between the probabilistic and deterministic method is very limited.

The following conclusion is summarized to answer the research question:

Cost-effective levee designs along the river Meuse in Limburg are robust in terms of height and are designed with the requirements of the new safety philosophy. LCCA is an appropriate tool to investigate levee design alternatives on a cost-effective manner. Accounting project investments only lead to an unfair alternative analysis. Including uncertainties in hydraulic boundary conditions during the alternative analysis does not lead to an other outcome. These uncertainties do contribute to the investigation of the optimal levee height robustness.
Glossary

**Benefits**
A positive growth in economical welfare (e.g. flood risk reduction), which often involves a comparison between two situations.

**Cost-effectiveness**
A manner to indentify the economical effectiveness of a purpose and/or project.

**Cutoff**
A pass trough a levee which can be enclosed by a demountable or dynamic system (often a stop-logg system).

**Decimation height**
The water level variation related to an increase or decrease of the recurrence interval with a factor 10.

**Depreciation**
The decrease in value of a project within a certain time.

**Discount rate**
Factor reflecting the time value of money that is used to convert cash flow occurring at different times to present time.

**Failure mechanisms**
Mechanisms leading to failure of a flood defence system.

**Future expenses**
All expenses incurred after occupation of a project.

**Initial expenses**
All expenses which are directly made (within a certain time frame).

**Length-effect**
The phenomenon that the probability of failure of a flood defence system increases with an increasing length.

**Life-Cycle Cost Analysis (LCCA)**
An economic evaluation technique that determines the total cost of constructing, owning and operating a project over a period of time. Some methods include benefits as well.

**Major maintenance**
Renovating and repair of building or infrastructural components in a system.

**Mound**
An earthy mass projecting above the surface.

**Multilayered safety**
A flood managing concept which distinguish measures to potentially decrease the impact of flooding into three layers: Prevention, spatial planning and crisis management.

**Net Present Value (NPV)**
LCCA method adding up the present value of the costs and benefits with help of a discount rate.

**Nominal discount rate**
Rate used to relate present and future money values taking into account the general inflation/deflation rate.

**Present value (PV)**
A value in where all (future) values are counted back to today's value used in several LCCA methods.
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<td><strong>Preventive maintenance</strong>  –  Preventief onderhoud</td>
<td>Regular maintenance like routine inspection or testing of a facility.</td>
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<tr>
<td><strong>Prob. of a flood</strong>  –  Overstromingskans</td>
<td>The probability that a flood occurs due to failure of the ring-levee or ring-levee section.</td>
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<tr>
<td><strong>Prob. of exceedance</strong>  –  Overschrijdingskans</td>
<td>The probability that a solution, in this case a flood, is equal or larger than a certain value.</td>
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<td><strong>Project costs</strong>  –  Project kosten</td>
<td>All costs up to completion of a project. (e.g. desing costs, construction costs, etc.).</td>
</tr>
<tr>
<td><strong>Purchasing power</strong>  –  Koopkracht</td>
<td>Value of a currency expressed in terms of amount of goods or services that one unit of money can buy.</td>
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<tr>
<td><strong>Reactive maintenance</strong>  –  Reactief onderhoud</td>
<td>Maintenance due to a problem or breakdown.</td>
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<tr>
<td><strong>Real discount rate</strong>  –  Reële discontovoet</td>
<td>Rate used to relate present and future money values not taking into account the general inflation/deflation rate.</td>
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<tr>
<td><strong>Recurrence interval</strong>  –  Herhalingstijd</td>
<td>The probability that an event will be equalled or exceeded in any given year.</td>
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<td><strong>Residual value</strong>  –  Restwaarde</td>
<td>Value assigned to a project investment at the end of the analysis period.</td>
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<tr>
<td><strong>Ring-levee</strong>  –  Dijkring</td>
<td>A system of dikes or high grounds enclosing a certain area in order to protect against flooding.</td>
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<tr>
<td><strong>Ring-levee section</strong>  –  Dijkvak</td>
<td>Part of the ring-levee with equal strength- and load characteristics.</td>
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<tr>
<td><strong>Robustness</strong>  –  Robuustheid</td>
<td>Accounting for future developments and uncertainties into the design, such that it keeps its function without radical changes and expensive measures during its whole lifetime with the ability to upgrade it, when economical attractive.</td>
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<tr>
<td><strong>Room for the River</strong>  –  Ruimte voor de Rivier</td>
<td>A programme of the Dutch department of Public Works and Water Managements to limit the risk of flooding thru the rivers in the Dutch delta.</td>
</tr>
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<td><strong>Salvage costs</strong>  –  Bergingskosten</td>
<td>The value of a project after the end of its assigned lifetime (consist of residual and servicable values).</td>
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<tr>
<td><strong>Water act 2009</strong>  –  Waterwet 2009</td>
<td>A Dutch law combined in 2009 of eight older laws where the probability of exceedance is embedded.</td>
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Introduction

1.1. Practical background

After the floods of 1993 and 1995 in the Netherlands, flood risks in Limburg (a province of The Netherlands) became an important discussion. After the floods of 1993, commission Boertien I (Boertien et al. 1994) advised the Dutch government in December 1994, to create more room for the rivers and additionally to construct a levee system. At that time the expectations were that flood risks could be reduced relatively fast by giving more room to the river. These space measures should finally guarantee a protection level of 1/250 years (advice commission Boertien II).

The floods of 1995 created an enormous extra political pressure to take necessary and preliminary measures. Therefore temporary levees were created in 1996 providing a preliminary solution for the flood hazards the area faced, giving a protection level of about 1/50. These temporary levees should be removed in about 10 years after completion of the room-measures. Unfortunately, these measures were not realistic (financial and technical) and did not resulted into the expected drop of the water levels. So that the preliminary levees turned up to be permanent, but were never dimensioned as such.

Each flood defence system along the river Meuse needs to be provided with a safety level of 1/250 years. Therefore, the Dutch Peel and Maasvallei water board, needs to upgrade their levee design to a 1/250 protection in a program for the coming decade.

Nowadays, a new discussion raised to change the safety norms. In the last few decades the safety norms were adjusted (Commission Becht (1972), Commission Boertien I (1992) and Commission Boertien II (1994)) (AcW 2012). In 2009, the new Water Act embedded the earlier stated probability of exceedance of 1/250 of the river Meuse in Limburg. Present thoughts, concerning the safety philosophy are expressed as acceptable probability of failure of the flood protection system as a whole, the so called “probability of a flood”. Future levee designs has to fulfil this new safety philosophy.

1.2. Problem definition

The water board need to construct levees up to a 1/250 safety level given the knowledge of the new safety level according to the new flood probability. The impact to the levee design is uncertain, although the belief exist that levees along the river Meuse in Limburg have to be reinforced (Kind 2011, de Groot 2012, VNK 2012).

Additionally, the water board is facing many more problems, such as limited space or an intense infrastructure. Before the disasters of 1993 and 1995, no levee systems were present yet. The landscape was already shaped with many buildings built along the river Meuse and an intense infrastructure, both above as underground. The temporary levees were constructed relatively fast in peoples backyards, on farmlands, crossing roads with a cutoff (Dutch: Coupure) and on top of cables and pipelines. As a result, little space is available for inspection and maintenance or to upgrade the levees in height or strength. Figure 1.1 shows a levee system in Arcen together with its problems.
Hence, there was and is not much space available to create a levee-system. Stakeholders like municipalities, citizens, companies and landscape organizations were, and are still, not used to have such flood defence system nearby. "Not in my backyard" is a famous statement indicating the acceptance of a spatial planning change is willingly by stakeholders until it affect their own values.

The new safety approach requires a more imposing design. After a certain time, ring-levees which will be constructed in the upcoming few years, need to be adjusted in order to fulfil the new safety philosophy requirements. Therefore, it seems logical to design the levees with certain robustness in which the levees can be upgraded relatively easily to the new safety approach. However, the relative importance, which can be defined in terms of benefits, of such upgrade is still unknown.

Uncertainties are present, influencing future investments. Climate change for instance, may increase the discharge of the river Meuse significantly over time while future river widening projects are decreasing the expected water levels related to a certain discharge. If the water level increases faster as forecasted, the major maintenance costs will be larger, too. Therefore one searches for a levee design which adapts to adjustments easily on a cost-effective manner.

However, the amount of investments becomes more and more an important discussion regarding a levee design. In contrast to the past, including extra robustness into a levee design has to have better arguments.

Currently, one searches for a balance between robustness and life cycle costs including uncertainties in hydraulic boundary conditions, as well as the influence to the spatial area. Is it beneficial to design levees in accordance to the new safety philosophy directly? Or will levees with a shorter lifetime and easy to adapt future climate changes be better?

1.3. Objective

By considering the balance, as formulated in paragraph 1.2, during the alternative analysis of levee designs, other alternatives may become of interest. In the upcoming years, the water board’s objective is to construct several "robust" levees along the river Meuse on a cost-effective manner taking all requirements and preferences into consideration.

\footnote{Accounting for future developments and uncertainties into the design like an extra crest width or levee height.}
The objective of this MSc-thesis is to investigate which balance between levee investments and robustness is cost-effective during the levee design alternative analysis, specified for cases along the river Meuse in Limburg. Robustness within this balance is related to the uncertainties in future hydraulic boundary conditions which possibly establish a better decision making process in contrast to the present situation.

1.4. Research question

The main focus of the research is for levees in urban areas along the river Meuse, in Limburg. These type of levees share the same history (i.e. historical flooding, safety levels, change to primary levees, limited space, etc.) and are dealing with problems like the limited space, dense infrastructure and many stakeholders. Research questions have been formulated in order to define the objective in real terms. The main research question for the MSc-thesis is:

**What are cost-effective levee designs including uncertainties in hydraulic boundary conditions?**

1. Life-Cycle Cost Analysis, or LCCA, will be used to investigate the cost-effectiveness of levee design alternatives. Some alternatives enhance larger maintenance-, service- or demolition-costs than others. Besides, the benefit of a ring-levee as a whole may vary over the alternatives (e.g. when different locations are involved). This leads us to the following key-question:

**How can life-cycle costs of different levee types and alignments be compared equally?**

2. Parameters of the LCCA depending on future hydraulic boundary conditions, which are uncertain. Knowing and including these uncertainties probabilistically into the LCCA may obtain a different LCC-effectiveness outcome. This lead us to the following key-question:

**How large are the uncertainties in hydraulic boundary conditions?**

3. Having these uncertainties known, one searches for an approach to include them into the LCCA, along with variables such as investments, benefits and levee robustness. Leading us to:

**How can the cost-effectiveness be investigated given the uncertainties in hydraulic boundary conditions?**

4. Accounting hydraulic boundary conditions probabilistically may change the LCC differences between levee design alternatives. The following key-question investigate the importance of probabilistic method:

**What influence does the probabilistic method have on the levee design alternative analysis?**

5. Currently the design- and statutory assessment methodology of the new safety philosophy is under development. Furthermore levees along the river Meuse require the present annual safety standard of 1/250. Including robustness now, may prevent future maintenance measures. In order to investigate whether or not the levee design alternatives have to (partly) comprise both safety approaches directly, the next key-question is defined:

**How large is the robustness for a cost-effective levee design?**
1.5. Research outline
During the investigation to the cost-effective levee design alternatives, two case studies will be carried out. The first case study will be used during the research while the second one is used as validation.

The MSc research outline is the following:

Chapter 2 - Realistic alternatives
This chapter summarize the used approach of defining, designing and screening realistic levee alternatives. At the end of this chapter, several alternatives will be presented which will be further in the investigation.

Chapter 3 - Life-Cycle Cost Analysis
This chapter introduce the theoretical framework of the LCCA and its translation of the method to levee alternatives. Subsequently, realistic alternatives will be analysed deterministically on their LCC.

Chapter 4 - Cost-effectiveness optimization
Next, the uncertainties in hydraulic boundary conditions will be investigated in order to include them into the LCCA. A practical framework will be formed to create a levee optimization based on its LCC. Here the hydraulic boundary conditions are included probabilistically. A preliminary conclusions is given based on the outcome of case study Arcen.

Chapter 5 - Case study Venlo
Case study Arcen is used during the formation of the cost-effectiveness optimization procedure. Case study Venlo will be used to validate this procedure as it entails a different set of parameters.

Chapter 6 - Conclusion and recommendation
Here the research questions are answered. The conclusions and recommendations are formulated regarding the applied procedure, robustness and future research.
This chapter summarizes the approach of defining, designing and screening realistic levee design alternatives. The full approach, or "Toolbox" is given in appendix C. At the end of this chapter, several realistic alternatives are specified for case study Arcen which will be used further in chapters 3 and 4.

Levee design alternatives will be defined for case study Arcen. Subsequently the alternatives are designed and screened. The "best" alternatives derived from this approach are named to be realistic in order to make a clear distinction between the defined alternatives and those that will be analysed further within the report.

The presence of the various preferences by stakeholders makes it impossible to capture the definition of the "best" alternative properly. The resulting realistic alternatives from the formed approach score highest in contrast to other levee design alternatives, although unwanted aspects are present within these alternatives.

This chapter contains 6 paragraphs. First some information is given regarding the design process in general. Next, case study Arcen is introduced. Subsequently, the approach to define, design and screen levee design alternatives is given. the last paragraph contains the discussion or dialogue related to this chapter.

2.1. Design process

A design process is a continuous circular motion with assumptions, checks, calculations and reconsiderations. There are no appropriate fixed rules available regarding the design process, like arranged questions which need to be answered. Especially when it comes to defining and screening alternatives in where assumptions have to be adjusted frequently, or preferences have to be change (TAW 2003).

There are different ways to distinguish phases of the design process. Figure 2.1 illustrate an example of a process specified for a levee design alternative analysis. Commonly it starts with an idea or initiative in order to solve a (prospected) problem in the initiative phase (yellow). Levee design alternatives are formed, and analysed during the funneling phase (blue). Here they are screened related to the functional requirements, boundary conditions, stakeholder preferences and costs. During this stage levee design alternatives will be discussed by the water board in cooperation with other stakeholders.

At the end of the funneling phase one preferred alternative will be chosen which will be detailed in the planning phase (green). A detailed design will be developed by optimizing the preferred situation (e.g. via optimization of the exact location, materials, etc.), resulting in a design solution which can be realized in its last phase, the realisation (orange). Often adjustments to the design are made in the last stage due to minor errors in the design or contractor preferences.

Due to the many problems the water board faces (e.g. restricted budget, limited space, dense in-
2. Realistic alternatives

Figure 2.1: Design process in accordance with HWBP (2013), the vertical variance represent the number of alternatives while the level of detail of the levee design is illustrated horizontally.

Infrastructure, public opinion, uncertainties in hydraulic boundary conditions), there is a considerable need to come up with a design process in which "out of the box" levee design alternatives are formed. Therefore, one searches for an approach where a large number of alternatives can be defined (A to H), including alternative with a difference in levee position.

A multi-criteria analysis (MCA) is a common technique to screen levee design alternatives during the funneling phase (paragraph 2.5). All alternatives which fulfil the design criteria (e.g. minimum levee height), will be analysed with help of screening criteria in the MCA. A levee design alternative is favourable when most criteria in this screening phase have the best score.

This thesis research is focussed on the first part of the funneling phase (see red dashed rectangle in figure 2.1). The realistic alternatives (A, E, G) are formed by defining, designing and screening the levee design alternatives. Subsequently LCCA of the realistic alternatives will be carried out during this research. Conclusions and recommendation of this research are based upon the results of the LCCA. Furthermore it provide cost information which helps stakeholders during the decision making process of levee alternatives.

2.1.1. Safety approaches

Paragraph 1.1 introduces the new safety assessment (probability of a flood) which consider the reliability of the levee system including the so called "length-effect". Currently the present safety assessment have to be applied, associated with the probability of exceedance of a water level. The first levee design in Limburg will be based upon the present safety requirement in accordance of the Water act 2009 with an annual probability of exceedance of 1/250. After a certain time the design have to fulfil requirements of the new safety approach. Major maintenance have to ensure these requirements.

The delta program (DP2014) gives insight into the development regarding the new safety approach (or strategy). Soon, this program will draw up a preferential strategy for the area around the major rivers.

Levee designs along the river Meuse in Limburg encompass both safety philosophies. At the moment levees are designed and assessed with the probability of exceedance. Future levee designs are based on new safety approach. By including a sort of robustness into the design, the new safety approach can be applied easily, preventing the unfavourable situation to have large costs in a later stadium. According to ENW (2007), robustness regarding a levee design means:

"Accounting for future developments and uncertainties into the design, such that it keeps its function without radical changes and expensive measures during its whole lifetime with the ability to upgrade it, when economical attractive."

During this research robustness will be investigated per levee design alternative by optimizing the cost-effectiveness of the levees including the uncertainties in hydraulic boundary conditions. Robustness is
2.2. Introduction case study Arcen

Arcen is a small Dutch village in the province of Limburg along the river Meuse. Arcen has approximately 2,200 citizens and the main business activities are agriculture and recreation. Furthermore it includes some historical buildings such as the “Schanstoren” and castle Arcen (famous for its garden) as well as century old houses. Figure 2.2 shows an impression of Arcen along the river Meuse. Parallel to the river a walking path is visible from where one could see the levee structure crossing peoples backyards. At the North-, East- and South-side of the area, no levees are present as higher grounds forestall possible floods of the river Meuse (de Groot 2012). The floods of 1993 and 1995 have had an huge impact in Arcen. The flood waves entered gardens, houses and restaurants causing an enormous amount of economical damage. During the flood of 1993, over 1.500 citizens were evacuated in the municipality of Arcen and Velden. Today, approximately 20 years later, people still remember these days like yesterday. Some are still having their own inventions of demountable kitchens and flood-proof living rooms to prepare themselves against potential floods in the future.

The governing stakeholders of Arcen are: The water board, residents, ministry of Infrastructure and the Environment, restaurant/public house, municipality of Venlo, Cultural Heritage Agency and recreation organisations. Each stakeholder entails different preferences.

2.3. Defining alternatives

This paragraph summarizes the approach to define alternatives. In order to form “out of the box” levee design alternatives, all possible levee alignments options and levee construction option types are gathered. The levee types are subdivided into green- and structural options. Occasionally both are applied as is shown below:

\[
\text{Alternative} = \text{alignment option} + (\text{green option} \cup \text{structural option})
\]  

(2.1)

Approximately 650 meter of the total 5.000 meter ring-levee will be analysed in this research. Furthermore the ring-levee part is subdivided into two smaller sections with similar parameters such as the construction types, loads, functions, safety assessment and present situation.

Two governing cross-sections are chosen where the alternative analysis is based on, making the screening process easier. In reality more than two cross-section can be picked if preferred.

Four levee alignments are defined:

- Option 0: Present alignment;
- option 1: Change levee alignment to the river-side;
- option 2: Change levee alignment to the other side of the houses (at the street);
- option 3: Connect houses at the river-side.

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2. Realistic alternatives

**Figure 2.3**: Overview of the various levee alignment options of case study Arcen

**Option 0** is the present location. Changes to the river flow profile are very low and it has minimum impact to the licence policy¹.

The residents along the river are important stakeholders. The levee crosses their backyards and has impact to their spatial view. Increasing the levees might result in an even more limited view to the river as well as their own backyard. **Option 1** changes the levee to the river-side which establishes a better connection between the houses and their backyard or increases the spatial feeling when watching outside from their houses. Unfortunately it impacts the river flow profile.

In order to preserve the outlook to the river as much as possible **option 2** is introduced. The present height of the street at the other side of the houses can be increased. It entails the fact that the houses nearby the river are exposed against potential floods at a much lower level as the other side of the houses.

A more innovative approach is to connect houses together creating a multifunctional flood defence. **Option 3** is defined forming the levee alignment to this approach. Most houses will be part of the flood defence. Doors and windows have to be closed during extreme events. Furthermore the flood defence has to provide safety against piping.

Next all levee types are checked if they fit within the two governing cross-sections. Subsequently, alternatives are cancelled out of the investigation which do not make horse sense. All other functions around the flood defence have to be preserved. There are exceptions possible, like a levee alignment at the river-side. In order to equalize the river flow profile, one has to account a compensation somewhere else in the same river cross-section. In the case of Arcen this compensation can be achieved by lowering the embankment at the other side of the river. This criteria is included in paragraph 2.4.

Per levee option alternatives are formed:

**Option 0: The same alignment**
Solutions within this levee alignment are:

A. Structural design option;
B. parallel green levee;
C. double wall system;

¹Some extra licenses have to be requested at the levee manager to perform activities along the levee like constructing, digging, chopping trees, etc.
D. double wall system with a structural design option;
E. green levee;
F. green levee with a single structural design option.

Option 1: Levee alignment at the river-side
Solutions within alignment 1 are:
G. Green levee;
H. green levee with a single structural design option;
I. double wall system;
J. green levee with a structural option;
K. structural design option.

Option 2: Levee alignment at the other side of the houses
The following solutions can be thought of for levee alignment option 2:
L. Double wall system;
M. dynamic system raising out of the street;
N. connecting houses including a structural design option against seepage.

Option 3: Connect houses at the river-side
There is one main solution which coincide levee alignment 3:
O. Connect houses, use sheet pile at the front side of the houses preventing seepage;

2.4. Designing alternatives
Design criteria are formed with the aid of various guidelines and rules of thumb. A sketch design of the defined alternatives is based upon these design criteria and included in the governing cross-sections. A summery of these criteria, consisting of function requirements, boundary conditions and design requirements is shown below. A more elaborate explanation is given in appendix C. A distinction is made between criteria regarding the functions of the levee, the dimensions of the levee, its strength and requirements during maintenance.

Some levee design alternatives may not fit within the governing cross-sections due to a lack of available space. After designing the levee alternatives into the governing cross-sections, this available space will be checked (paragraph 2.4.1).

Levee functions
- Levee alternatives will be designed based on the present safety approach;
- design has to fulfill a minimum protection level corresponding with an annual probability of exceedance of 1/250;
- design has to fulfill a protection level corresponding with an annual probability of exceedance of 1/1250 after major maintenance;
- a physical maximum discharge of 4.600 m$^3$/s is assigned for the river Meuse in Limburg;
- all existing infrastructural functions in, on top and along the levee have to be preserved;
- the reduced area of the river flow profile due to a change in levee alternative, have to be compensated with the same area somewhere in the same cross-section;
- monumental buildings and structures have to be preserved.

Levee dimensions
- Crest freeboard of 0,50 meter;
- a minimum crest width of 4,5 meter including a maintenance and inspection path;
- a minimum crest width of 3,0 meter without a maintenance and inspection path;
- a maximum structure height of 0,80 is allowed with respect to the surface in urban areas to preserve the ability to have a proper view on the river.
2. Realistic alternatives

Strength of the levee

- The probability of exceedance calculations are based upon the failure mechanism overtopping and overflow;
- a clay layer with erosion type 2 and a thickness of 1,0 meter will be applied on the crest and slope/berm on the river-side of a green levee;
- a 5,0 meter long clay layer of 1,0 meter thick is required to decrease the probability of failure due to piping unless a structural element like a sheet pile is present;
- sheet pile wall will be used for all retaining wall types with optional a concrete capping beam;
- the total length of a cantilever wall is three times the retaining height.

Maintenance

- The preferred maintenance and inspection path position is on top of a green levee or at the land-side of a structural levee alternative;
- the maintenance and inspection path has a width of 3,0 meters;
- a maintenance and inspection path has to be accessible during the governing high water event;
- the maximum distance between the maintenance path and a structural alternative is 2,0 meter.

2.4.1. Available space

Appendix A.3 illustrates the alternatives drawn in the governing cross-section. In some cases it is clear that an alternative doesn’t fit. There can be thought of measures which enable this alternative within the available space. But, if it ends up looking a lot like an other alternative, it can be excluded as realistic alternative. Table 2.1 shows the results of both levee sections. A "No" is assigned to alternatives not fitting within the available space when they fulfill all design criteria. Other alternatives are assigned with a "Yes".

Table 2.1: Results of the available space check of both levee sections case study Arcen

<table>
<thead>
<tr>
<th>Alternatives</th>
<th>Section 1</th>
<th>Section 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Structural design option</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>B. Parallel green levee</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td>C. Double wall system</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>D. Double wall system with a structural design option</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>E. Green levee</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td>F. Green levee with a single structural design option</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>G. Green levee</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td>H. Green levee with a single structural design option</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td>I. Double wall system</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td>J. Green levee with a structural option</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td>K. Structural design option</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>L. Double wall system</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>M. Dynamic system raising out of the street</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>N. Connecting houses including and structural design</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>O. Connect houses and sheet pile</td>
<td>Yes</td>
<td>Yes</td>
</tr>
</tbody>
</table>

2.5. Screening alternatives

This paragraph shows the result of the screening procedure for case study Arcen. A multi-criteria analysis has been performed consisting of various screening criteria (e.g. Hydraulic impact, costs, constructibility). Subsequently all criteria consist of multiple aspects. The scoring of these criteria c.q. aspect are based upon points. For instance, executing a sheet pile at an angle creates more difficulty in the construction phase. This aspect lead to more points in the constructibility criteria. The sum of all points per criteria will be evaluated in accordance with the following:

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2.5. Screening alternatives

Each alternative is assigned with good (+), doubtful (0) or bad (–) per screening criteria. One seeks for a point system which results in a large variety between the alternatives outcome. A good score indicates "no too not much" influence of the criteria though the alternative. A bad score indicate an unwanted situation. For instance, if the hydraulic impacts becomes significant. Notice the subtle assessment of the points. The resulting multi-criteria analyses of case study Arcen are shown in tables 2.2 and 2.3.

Furthermore stakeholder points are assigned to distinguish the mutual difference between importance of the screening criteria. The stakeholder points are assigned fictively with aid of the "project and safety" group of the water board with help of the Delphi-method.

Table 2.2: MCA ring-levee 65 Arcen, section 1

<table>
<thead>
<tr>
<th>Screening criteria</th>
<th>Alternatives</th>
<th>Stakeholder Points</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>Hydraulic impact</td>
<td>+</td>
<td>0</td>
</tr>
<tr>
<td>Costs</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>Constructibility</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Maintenance &amp; inspection</td>
<td>-</td>
<td>0</td>
</tr>
<tr>
<td>Monuments</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>Nature</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>Hindrance usage</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>Cables and pipes</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>Trees</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>Connection sections</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>Hindrance execution</td>
<td>+</td>
<td>+</td>
</tr>
</tbody>
</table>

Table 2.3: MCA ring-levee 65 Arcen, section 2

<table>
<thead>
<tr>
<th>Screening criteria</th>
<th>Alternatives</th>
<th>Stakeholders</th>
<th>Points</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
<td>B</td>
<td>C</td>
</tr>
<tr>
<td>Hydraulic impact</td>
<td>+</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Costs</td>
<td>0</td>
<td>+</td>
<td>0</td>
</tr>
<tr>
<td>Constructibility</td>
<td>+</td>
<td>+</td>
<td>0</td>
</tr>
<tr>
<td>Maintenance &amp; inspection</td>
<td>-</td>
<td>0</td>
<td>+</td>
</tr>
<tr>
<td>Monuments</td>
<td>+</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>Nature</td>
<td>+</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>Hindrance usage</td>
<td>0</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>Cables and pipes</td>
<td>+</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>Trees</td>
<td>+</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>Connection sections</td>
<td>+</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>Hindrance execution</td>
<td>+</td>
<td>+</td>
<td>0</td>
</tr>
</tbody>
</table>

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2.5.1. Results of Alternative analysis
Alternatives are defined by combining levee alignments with one or several levee types. Subsequently a sketch design is established based on several design criteria. The levee design alternatives are checked on their available space within the governing cross-section. At last a MCA is formed based on several screening criteria.

The MCA shows that the alternatives with levee alignment 2 (Levee alignment at the river side) and 3 (Connect houses at the river side) score to bad and/or doubtful for the case study Arcen.

According to the screening procedure the alternatives C, D and K are most attractive regarding levee section 1.

The alternatives B and E are most favourable in section 2, having the alternatives C, F and K as a close second place.

There are two alternatives; K and C, which are in both sections of interest. The alternatives D (section 1) and E (section 2) have corresponding levee types and are therefore both assigned as realistic alternatives, too. Alternative B in section 2 is the most favourable one and will also be analysed further. An overview of the realistic alternatives is given below in table 2.4 and figure 2.4.

<table>
<thead>
<tr>
<th>Screening criteria</th>
<th>Alternatives</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section 1</td>
<td>C D K</td>
</tr>
<tr>
<td>Section 2</td>
<td>B C E K</td>
</tr>
</tbody>
</table>

2.6. Discussion
One has defined, designed and screened levee alternatives as fair as possible. A multi-criteria analysis is an often applied technique to analyse levee alternatives in this design stage. Design- and screening criteria are formed with limited to no communication with stakeholders. A change in these criteria as a result of other requirements or stakeholder points could lead to other realistic alternatives.

The Dutch strategy; “Rivers widening when possible, increasing levees when necessary” seems to be conflicting with the levee alignment at the river side. Why would one first plan river widening projects, and afterwards limiting its positive impact by moving levees into the river embankments? However, by implementing the design criteria that the river flow profile have to be at least the same size as the present situation, one obtains a fair alternative. Hence, levee alignments at the river-side require a compensation measure elsewhere in the same profile. These costs are part of the levee alternative. If a levee design alternative turns out to be the cheapest including the widening measure, it still is a realistic solution.
2.6. Discussion

(a) Case study Arcen: Realistic alternative 1C
(b) Case study Arcen: Realistic alternative 1D
(c) Case study Arcen: Realistic alternative 1K
(d) Case study Arcen: Realistic alternative 2B
(e) Case study Arcen: Realistic alternative 2C
(f) Case study Arcen: Realistic alternative 2E
(g) Case study Arcen: Realistic alternative 2K

Figure 2.4: Realistic alternatives case study Arcen
This chapter analysis the life-Cycle costs of the realistic levee design alternatives. Chapter 2 formulated all possible alternatives by combining a levee alignment option and a structural option. Furthermore design and screening criteria have been used, resulting in a small number of realistic alternatives for the case study Arcen. The analysis will be optimized by including a variable levee height combined with uncertainties in the hydraulic boundary conditions in the next chapter 4. Subsequently a second case study is carried out in chapter 5 followed by the conclusion and recommendations in chapter 6.

Life Cycle Cost Analysis or LCCA is a widely acknowledged technique as a decision tool originating from the early 1960’s. Nowadays, the technique has been applied among others in infrastructural, architectural and machinery fields (Jawad and Ozbay 2006). It can be used to evaluate alternatives during a design process. Via LCCA projects, or alternatives are investigated economically over their whole analysis period including project-, maintenance-, inspection- and end-of-lifetime costs. A literature review of LCCA is presented in appendix E.6 where a description is given of the cost terminology, analysis period and the calculation procedure. This chapter gives a theoretical framework how levee design alternatives can be analyses on their LCC.

There are various LCCA methods available and many can be used for levee design. The net present value (NPV) method is one of the often applied methods regarding levee design alternative which includes the benefits as well. Moreover the HWBP suggests to use the NPV method (van den Berg et al. 2013). This method encompasses the idea to require larger benefits than investments. Due to the fact that the NPV method is often applied, used by the HWBP and includes benefits, it will be used within this research. Equation 3.1 illustrate the NPV-method shortly.

\[ NPV = PV(B) - PV(I) \]  

Where:

- \( NPV \) : Net Present Value [€]
- \( PV \) : Present Value [€]
- \( B \) : Benefits of flood risk reduction [€]
- \( I \) : Investments [€]

The next paragraph introduces the benefits of a levee design. Paragraph 3.2 formulates the way how to include the investments into the LCCA. Additionally the NPV calculation is introduced followed by the paragraph of the case study Arcen. Subsequently a discussion is reported in paragraph 3.5.
3.1. Benefits of flood risk reduction

Some LCCA methods include benefits of a project or alternative. The benefits of a levee design alternative can be estimated by calculating the flood risk reduction between the present and the new situation. The main purpose for including benefits is to create a fair levee design analysis. A different levee alignment or levee height entails a flood risk difference between alternatives.

A safety increase of a ring-levee, decreases the probability a flood occurs. This results in a smaller flood risk and is therefore beneficial. The risk due to flooding caused by overflow and overtopping is considered as the only failure mechanism to estimate the benefits.

The benefits will be estimated over a finite analysis period of $T = 100$ years. Within this period there is a chance that one flood, multiple floods (e.g. the floods of 1993 and 1995) or no floods occur. The probability that one or multiple floods occur over a certain period is binomial distribution. In general, a discrete stochastic variable $Y$ has a binomial distribution with parameters $n$ and $p_f$, in where $n = 1, 2, \ldots$ and $0 \leq p_f \leq 1$, if $X$ have possible values $0, 1, \ldots n$ with chance (Dekking et al. 2005):

$$P(Y = n) = \binom{T}{n} p_f^n (1 - p_f)^{T-n} \quad (3.2)$$

Where:
- $p_f$ Annual occurrence probability of a flood [-]
- $Y$ Stochastic variable [-]
- $n$ Number of floods during time interval $T$ [-]
- $T$ Analysis period [year]

If the risk (i.e. the product of the flood damage and the annual probabilities of failure) of each flood is independent, the expected value of the number of floods within the analysis period is:

$$E[Y] = p_f T \quad (3.3)$$

However, if a second flood occurs shortly after the previous one, a lower flood damage is expected. Besides the increased awareness of citizens, the land is not fully recovered yet, leading to less damage. Hence, whenever this dependency between floods is significant, the expected value of the binomial distribution (3.3) cannot be applied. The second flood risk have to be reduced due to the decreasing flood damage. In this thesis floods are assumed to be fully independent, hence equation 3.3 is applicable.

The flood risk can be calculated by multiplying the probability of levee failure with expected damage caused by that flood:

$$R_a = p_{f,a} D_a \quad (3.4)$$

Where:
- $R_a$ Risk of flood event $a$ [€]
- $p_{f,a}$ Annual occurrence probability of flood event $a$ [-]
- $D_a$ Expected damage of flood event $a$ [€]

Each flood is related to a certain expected damage. If the probability of failure drops due to a higher discharge, the expected damage increases. With aid of a Q-h relation this damage can be related to the discharge ($D(Q)$).

The flood probability equals the exceedance probability of discharge $Q_a$:

$$p_{f,a} = P(X \geq Q_a) \quad (3.5)$$

Let the stochastic variable $X$ follow a probability density function $f(x)$. Then the probability of failure can be found by integrating $f(x)$ from discharge $Q_a$ to infinity. Combining the integral with equations 3.3 and 3.4, lead us to the total yearly risk of flooding:

$$R = \int_{Q_a}^{\infty} f(x) D(x) \, dx \quad (3.6)$$
### 3.1. Benefits of flood risk reduction

The yearly risk reduction between the present and new situation can be calculated by:

\[
B = R_a - R_b = \int_{Q_a}^{\infty} f(x) D(x) \, dx - \int_{Q_b}^{\infty} f(x) D(x) \, dx
\]  

(3.7)

Where:
- \(B\) = Economical benefits of flood risk reduction within a ring-levee [€]
- \(R_a\) = Yearly flood risk of the present situation within the analysis period [€]
- \(R_b\) = Yearly flood risk of the new situation within the analysis period [€]
- \(Q_a\) = Minimal discharge causing failure of the flood defence with the present situation [m³/s]
- \(Q_b\) = Minimal discharge causing failure of the flood defence with the new situation [m³/s]

Hence, the benefits of a levee design alternative can be estimated by calculating the risk reduction with help of equation 3.7.

Theoretically the analysis period \((T)\) of the LCCA is infinite. Practically, it is often limited to 100 years which is in line with the conceptual report of HWBP (2013).

The flood damage \((D(x))\) and the probability density function \((f(x))\) of the annual probability of failure \((p_f)\) in equation 3.7 are elaborated in the next two paragraphs.

#### 3.1.1. Expected damage

There are various methods to estimate the economical damage of a flood event. The preferred method depends on the data available, the effort to elaborate such calculation (i.e. time and money) and the required accuracy of the outcome.

Benefits between alternatives may differ due to a difference in the probability of failure and economical damage. Having overflow and overtopping chosen as the only failure mechanism, benefits are affected primarily by the levee height. Each alternative may have its own optimum levee height entailing a different probability of failure.

Secondly, the levee alignment differs over the alternatives leading to a difference in flood damage. A fictive example of various alternatives in where the levee alignment differs is shown in figure 3.1. The circle indicate the protected area The marked areas are protected. Levee design alternatives 0, 1 and 2 have the same levee alignment as the present situation. The levee alignment in alternatives 3 to 10 differs.

![Figure 3.1: Illustration of a difference in levee alignment leading to a variation in economical flood damage of alternatives in a ring-levee. The marked areas are the protected areas](image)

One way to estimate the flood damage is by using a (social) cost-benefit analysis wherein present benefit data per ring-levee is estimated. Another manner is to use the standard method damage and casualties which is implemented in the so called HIS-SSM software\(^1\) (Kok et al. 2006). This method

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\(^1\)HIS-SSM: Hoogwater Informatie Systeem - Schade en Slachtoffer Module

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introduce various categories. Per unit of the category a maximum damage is assigned. Depending on the damage factor, which is related to the category and inundation height, the total economical damage can be estimated. More manners can be thought of like using various maps and GIS-software\(^2\).

During this research data is used from de Groot (2012) which present the results of a HIS-SSM analysis. Results are shown in table 3.1 and the related graph in figure 3.2.

The economical damage and casualties are estimated at four different water levels (H-1\(F_D\), H, H+1\(F_D\), and H+2\(F_D\)) in where \(F_D\) represents the so called decimation height. This height equals the height difference associated to a difference of the flood probability with a factor 10. H is the water level equal to the present statutory assessment. The economical damage for a certain flood probability can be estimated by interpolating and extrapolating these values.

<table>
<thead>
<tr>
<th>Water levels</th>
<th>Probabilities</th>
<th>Economical damage</th>
<th>Casualties</th>
</tr>
</thead>
<tbody>
<tr>
<td>H-1(F_D)</td>
<td>1/25</td>
<td>€ 40 million</td>
<td>0-5</td>
</tr>
<tr>
<td>H</td>
<td>1/250</td>
<td>€ 65 million</td>
<td>0-5</td>
</tr>
<tr>
<td>H+1(F_D)</td>
<td>1/2.500</td>
<td>€ 90 million</td>
<td>0-20</td>
</tr>
<tr>
<td>H+2(F_D)</td>
<td>1/25.000</td>
<td>€ 140 million</td>
<td>0-70</td>
</tr>
</tbody>
</table>

The economical damage as well as the amount of casualties are given. From a societal point of view, there is a significant influence to the number of casualties, but rather difficult to add them as benefits within the LCCA. Many literature use both values as indicator for the total damage instead of combining them to one value (RIVM 2004). Within this research, differences in casualties over the levee design alternatives are not included.

A trend-line is added in figure 3.2 which represent the economical damage at each flood probability. Although it presents the accuracy in relation to the actual values well, this damage does not have to be that continuous as it is suggested here. For instance, the damage due to the difference between the inundation heights of 0,1 and 0,6 meter is larger than the inundation height between 1,5 and 2,0 meters inside a living room. Another example is an extra threshold within the ring-levee which inundate part of the ring-levee only at a certain height. One condition of this function is certain; the flood damage increases with a decreasing flood probability.

For the case study Arcen the continuous line seems to be appropriate. The area which is inundated does not change over the flood probability much (de Groot 2012) which indicates that there is no extra threshold within the ring-levee. Besides, most houses in Arcen are not that high the surface level of Arcen increases gradually.

### 3.1.2. Probability of failure

Many mechanisms leading to failure of a levee system. For instance, instability of a structure lead to a breach in the flood defence. In our model, overtopping and overflow are assumed to be the only mechanisms causing failure of the ring-levee. Subsequently the future safety philosophy will be assessed by applying the standard statutory assessment. Hence, in our model, the ring-levee fails if a water level exceeds the required threshold. For instance, if a levees annual probability of failure is 1/250, a 1/251 water level cause failure.

Failure of a levee does not necessarily cause large damage. In fact, there is a wide range of economical damage over the failure causes. While a levee breach leads to inundation of a large area, severe overtopping cause failure too, but its economical impact is limited. Hence, an assumption has

\(^2\)GIS software: Geographic information systems software which encompasses a broad range of applications in where various digital data-sets and/or maps are sorted geographically.
3.1. Benefits of flood risk reduction

to be made what encompasses failure: As soon as a water level becomes larger than the given probability of exceedance threshold, total failure occurs related to the economical damages as stated in paragraph 3.1.1 equal to the rivers water level.

Yearly maximum discharge data (D.5) is used within a fitting program (Easyfit) in order to find the appropriate distribution \( f(x) \) in equation 3.7. Figure 3.3 illustrates a histogram with the used data and a Gumbel-max distribution which is one of the extreme value distributions. It is an often applied distributions for this purpose (de Wit 2004, Jansen 2007) and fits best according to the fitting program. The Gumbel-max probability density function is given in equation 3.8:

\[
f(Q) = \frac{1}{\beta} \exp \left( -\frac{Q - \mu}{\beta} - \exp \left( -\frac{Q - \mu}{\beta} \right) \right) \quad (3.8)
\]

Where:
- \( f(Q) \): Gumbel-max probability density function with variable \( Q \) [-]
- \( Q \): Discharge \([m^3/s]\)
- \( \mu \): Mode parameter (1276,1) [-]
- \( \beta \): Scale parameter (416,44) [-]

Casualties are not included within this analysis leading to an underestimation of the benefits. In contrast, the probability of failure is defined in such a manner that it leads to an overestimation of the benefits.
3.2. Investments

The investments can be calculated by accounting all present values of a project up to the end of the analysis period. The costs can be divided into different categories as can be seen in equation 3.9. Each category is elaborated in Appendix E.6. The total investments, neglecting discounting can be expressed by:

\[ I_{\text{tot}} = I_{\text{pr}} + I_{\text{Ma}} + I_{\text{Se}} + I_{\text{En}} \]  

(3.9)

Where:

- \( I_{\text{tot}} \): Total investment [€]
- \( I_{\text{pr}} \): Project investment [€]
- \( I_{\text{Ma}} \): Maintenance and inspection costs [€]
- \( I_{\text{Se}} \): Service costs [€]
- \( I_{\text{En}} \): End of life costs [€]

Project investments are all expenses up to completion of the levee reinforcement. The maintenance costs are often divided by preventive (i.e. inspection, renewing rubber strips, etc.), reactive (i.e. repairing of an unexpected damage) and major maintenance (reinforcing a levee):

\[ I_{\text{Ma}} = I_{\text{pm}} + I_{\text{rm}} + I_{\text{mm}} \]  

(3.10)

Where:

- \( I_{\text{Ma}} \): Total maintenance and inspection investments [€]
- \( I_{\text{pm}} \): Preventive maintenance investments [€]
- \( I_{\text{rm}} \): Reactive maintenance investments [€]
- \( I_{\text{mm}} \): Major maintenance investments [€]

The service costs can be seen as the energy or fuel costs, as well as the use of a stock\(^3\), or the

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\(^3\)Some levee types involve demountable elements which are stocked elsewhere. These systems can be installed during high water.

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mobilisation costs during an extreme event. The end-of-life costs refer to the demolish cost including the decrease of recycled material (residual value) and represent its remaining life (serviceable life value).

The sunk costs represent a value which is irrelevant for the LCCA and is less straightforward than the other costs categories. The sunk costs should not be included into the LCCA.

On occasion the lifetime of a realistic levee design alternative does not end at the same analysis period. In these cases the residual value at \( t = T \) is calculated with a linear depreciation.

Having the investments introduced shortly, estimations of these values are often made with help of some standardized indicators (e.g. GWWkosten.nl, KOSWAT) providing general civil technical cost-data. Agencies and governmental organisations often have their own indicators as well.

The investment estimations of the alternatives are based on the cost indicator "GWWkosten.nl". The workload is estimated on expert judgement. Appendix G shows the valuation of the investments per realistic alternative.

### 3.3. NPV calculation

All investments and benefits have to be adjusted to their present value (PV). This can be done by using a real discount rate and an economical growth rate as shown in equations 3.11 and 3.12.

\[
PV(B) = \sum_{t=0}^{T} \frac{B}{(1 + r - g)^t} \tag{3.11}
\]

\[
PV(I_{tot}) = \sum_{t=0}^{T} \frac{I_t}{(1 + r - g)^t} \tag{3.12}
\]

Where:
- \( B \) Benefits in the analysis period \([\text{€}]\)
- \( I_t \) All Investments at time \( t \) \([\text{€}]\)
- \( T \) Analysis period \([\text{Year}]\)
- \( t \) Time \([\text{Year}]\)
- \( r \) Real discount rate \([-]\)
- \( g \) Economical growth rate \([-]\)

The real discount rate adjusts future expenses to their present value. Often the discount rate is nationally assigned. The Dutch government uses a 2,5% real discount rate with an additional "robustness factor" of 3% (Rienstra and Groot 2012). This robustness factor can be included if future investments and changes are uncertain. Within this research one accounts for future uncertainties partly (at least the hydraulic boundary conditions uncertainties). For this reason the robustness factor is not included into this analysis directly. At the end the real discount rate will be varied to investigate its dependency within LCCA.

The economical growth can be estimated as the average percentage increase of the Gross Domestic Product\(^4\) or GDP. According to the MKBA of Kind (2011), the Transatlantic Markets scenario defined a growth-percentage of 1,9%. However, the MKBA report of Deltares also described a sort of maximum in economical growth of about 50%. They include economical growth by increasing the water level over time. Although, one does not analyse certain prospects to future increases in welfare, the maximum of 50% seems reasonable. This maximum economical growth of 50% results in a growth of around 0,4% per year. The growth-percentage of 1,9% is too much in relation to the 2,5% discount rate. Moreover, the economical growth could be estimated by considering the particular area within the ring-levee and checking for future plans. Currently there are no large construction projects scheduled

\(^4\text{Dutch: Bruto binnenlands product (BBP)}\)

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within the ring-levee. Thus the 0,4% economical growth seems to be appropriate for the Arcen area and will be used during LCCA of the case study.

### 3.3.1. Ring-levee to ring-levee section

An important aspect within the NPV calculation is the inclusion of the benefits in relation to the quantity of the investments. The distribution of the benefits over a ring-levee has to be determined based on the relative importance per section. A levee section along a boulevard attracts more benefits than an other section within the ring-levee consisting of a green levee in an agricultural environment.

There are two main ways to calculate the investments and benefits correctly:

- Account investments and benefits of the whole ring-levee;
- Account investments and benefits of the ring-levee section;

In the latter way the total benefits have to be adjusted by the relative investments of that section. Hence, multiplying with the sectional investments and dividing by the total investments.

A decrease in risk within this analysis is caused by a decrease of the probability of overflow/over-topping. The whole ring-levee has to be increased to create a decrease of this failure probability. To warrant the requirement properly that the whole ring-levee will be reinforced rather than just a section, the first calculation way is chosen.

The investments of the other sections within the ring-levee will be estimated by a general price indicator according to table 3.2. Prices changed due to inflation. However, the detailed scale of these figures is very limited and therefore involves a large error.

Table 3.2: Cost indicators of the Dutch programme for levee strengthening projects along the river Meuse in Limburg (Sluitstukkaden 2009)

<table>
<thead>
<tr>
<th>Levee categories</th>
<th>Measure</th>
<th>Project costs</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Green levee</td>
<td>heighten: 0,00-1,00 m</td>
<td>1.120 €/m</td>
<td></td>
</tr>
<tr>
<td>Green levee</td>
<td>new or heighten &gt; 1,00 m</td>
<td>2.210 €/m</td>
<td></td>
</tr>
<tr>
<td>Green levee</td>
<td>strengthen</td>
<td>1.050 €/m</td>
<td></td>
</tr>
<tr>
<td>Structural levee</td>
<td>heighten: 0,00-1,00 m</td>
<td>820 €/m</td>
<td></td>
</tr>
<tr>
<td>Structural levee</td>
<td>new or heighten &gt; 1,00 m</td>
<td>9.120 €/m</td>
<td></td>
</tr>
<tr>
<td>Structural levee</td>
<td>strengthen</td>
<td>4.970 €/m</td>
<td></td>
</tr>
</tbody>
</table>

### 3.4. Case study Arcen

All realistic alternatives are budgeted for the case study Arcen in appendix G. Within this paragraph the LCCA will be carried out deterministically with the minimum required levee heights correlated to an annual probability of failure of 1/250. The present annual probability of failure of ring-levee 65 is 1/25 (de Groot 2012).

The lengths of sections one and two are approximately 100 and 550 meter. The length of the other levees is approximately 4350 meter.

The outcome, of the investments per alternative per meter is shown in table 3.3. Subsequently the total investments, benefits and net present values are given, too.
3.5. Discussion

Life-cycle costs of different levee design alternatives can be considered equally by calculating the NPV with inclusion of the present value of both the levee investments and the benefits.

Theoretically a levee design alternative is only economically acceptable if the NPV is positive. Shortly it says that the benefits are larger than the investments of a levee within the analysis period. However, a ring-levee consist of various types and locations of levees. A levee through a boulevard has more aesthetic importance with respect to a green levee in an agricultural environment. Although an alternative with a negative NPV should be a warning, it does not necessarily require to delete this alternative as possibility under all circumstances.

The benefits can be approximated by estimating the expected flood risk within the LCCA period. The flood risk depends partly on the flood damage which can be estimated if the flood risk is assumed to be independent. However, this dependency is not that straightforward. It takes years to re-establish a flood area. Practically, a second flood within several years causes relatively less damage. Furthermore a flood has a large societal impact, often lead to a redevelopment of the flood defence as has been the case after the floods of 1993 and 1995.

Using a social cost-benefit analysis gives a primary estimate of the economical damage, but is limited in accuracy. Within this model casualties were not included. If there is a significant difference in benefits between alternatives due to the variety in levee alignment, one should consider to include the number of casualties, too.

In general LCCA enhances multiple assumptions and estimations. One tries to achieve an as fair as possible analysis with a relative small error. However, the actual error within the analysis is very hard to determine. Besides the uncertainties in hydraulic boundary conditions, which will be investigated in the next chapter, there are many more unknown aspects. For instance, the height of the discount- and economical growth-rate or political developments regarding the future safety philosophy. Most important within the analysis is the relative difference between the alternatives. If it turns out that a parameters is overestimated, the effect is limited if it applies to all alternatives.

Preferences of stakeholders are increasing, especially in Limburg where the importance of levees changes (secondary levees became primary). Thoughts are to separate the costs of a levee reinforcement between governmental stakeholders in order to increase the aesthetic value. Here, one has to keep in mind that the resulting LCC do not illustrate the total levee costs and benefits properly. A better way is to consider the relative difference between the levee design alternative LCC combined with the actual cost of the final design.

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4

Cost effectiveness optimization

Realistic levee design alternatives are formed for the case study Arcen in chapter 2. LCCA is introduced in chapter 3. In this chapter the levee height is optimized by finding the optimum NPV including the uncertainties in hydraulic boundary conditions. Next, the optimization procedure will be used for case study Venlo.

The chapter starts with the basics of the optimization procedure. Next a practical interpretation of the earlier stated benefits and investments are given in order to include them into the optimization procedure. The uncertainties in hydraulic boundary conditions are summarized within this chapter, too. A full analysis of the hydraulic aspects both deterministic and probabilistic are given in appendix D. Subsequently, the results of the optimization procedure are given together with the outcome of the deterministic LCCA. A preliminary conclusion is given of the probabilistic and deterministic calculations These in paragraph 4.4 followed by some discussion aspects.

4.1. Optimization procedure

The aim of the optimization procedure is to find the optimum levee height per alternative when it comes to the NPV given the uncertainties in hydraulic boundary conditions. This optimum can be found by seeking the argument ($\Delta h_1$) for which the given $NPV$-function attains its maximum value (4.1). The minimum required levee height ($h_{min}$) is related to the deterministically determined height corresponding with an annual probability of exceedance of 1/250. Hence, all alternative require a minimum levee height related to the present statutory assessment. The extra height ($\Delta h_1$) is the optimization parameter within the procedure, reflecting the robustness of the levee height. A large robustness result in larger project investments, but less major maintenance costs and larger benefits.

$$h_1 = h_{min,1} + \arg \max_{\Delta h_1} NPV$$

Where:

- $h_1$: Most cost effective levee height at $t=0$ [m N.A.P.]
- $h_{min,1}$: Minimum required levee height at $t=0$ [m N.A.P.]
- $\Delta h_1$: Variable height above the minimum required levee height at $t=0$ [m]
- $NPV$: Net present value [€]

Chapter 3 set out the way how to calculate the NPV including benefits and investments. These two parameters influence the variable levee height ($h_1$) and the reinforcement height ($h_2$) within the optimization procedure as can be seen in equation 4.2. Both parameters influence the benefits. The project investments depending on the variable levee height ($h_1$) while major maintenance varies over the reinforcement height ($h_2$).
\[ NPV = PV\left(B_1(h_1)\right) + PV\left(B_2(h_2)\right) - PV\left(I_{pr}(h_1)\right) + PV\left(I_{mm}(h_2)\right) + PV\left(I_{Se}\right) + PV\left(I_{pm}\right) + PV\left(I_{rm}\right) + PV\left(I_{En}\right) \] (4.2)

Where:

- \( PV \) Present value [€]
- \( B_1 \) Benefits in the period between completion and major maintenance [€]
- \( B_2 \) Benefits in the period between major maintenance and end-of-life [€]
- \( I_{pr} \) Investments (project costs) [€]
- \( I_{mm} \) Investments (major maintenance) [€]
- \( I_{Se} \) Investments (service) [€]
- \( I_{pm} \) Investments (preventive maintenance) [€]
- \( I_{rm} \) Investments (reactive maintenance) [€]
- \( I_{En} \) Investments (end-of-lifetime costs) [€]
- \( h_1 \) Total levee height at \( t=0 \) [m N.A.P.]
- \( h_2 \) Total levee height after major maintenance [m N.A.P.]

The actual time to plan and execute major maintenance (\( T_{pm} \)) is variable depending on future hydraulic boundary conditions, lifetime of the levee or levee-elements, political aspects (e.g. financial budgets) and future spatial developments. Moreover, it could differ over the levee design alternatives. Planning major maintenance after 40 years to a levee alternative with a lifetime of 40 years seems logical.

The LCCA period is 100 years independent of the lifetime of the levee design alternatives. Within this research, a design choice has been made regarding the time between completion and major maintenance of half the analysis period (50 years). The expected lifetime of many levee elements is approximately 50 years.

The optimization procedure is graphically illustrated in figure 4.1. The three vertical bars representing the levee height during the start of the project (\( t=0 \)), major maintenance (\( t=T_{pm} \)) and at the end of the analysis period (\( t=T \)). The bottom part of each bar illustrates the height of the present situation. The minimum required levee height (\( h_{min} \)) in accordance with the present safety requirement is the next part of the bar on top of the present situation.

Figure 4.1: Schematic overview of the optimization procedure
The additional height ($\Delta h_1$) will be varied in order to find the most optimum NPV. A full calculation will be performed with a step-size of 0.10 meter. A calculation with the third step (0.30 meter) is illustrated in the schematic overview (4.1). In order to fulfil the new safety requirements at time $t = T_m$ and satisfy the new hydraulic boundary conditions at $t = T$ major maintenance may be executed. Hence, if the required levee height turns out to be larger as the situation of $h_1$, the levee height should be increased with the reinforcement height ($\Delta h_2$).

The expected benefits and investment from $t = T_m$ to $t = T$ will be estimated with the aid of a Monte-Carlo simulation. This calculation method simulate the uncertainties in hydraulic boundary conditions probabilistically. All simulations represent the related annual failure probability at $t = T$ which is 1/1250 for case study Arcen. By extracting the resulting height of each simulation ($h_2$) from the levee height before major maintenance ($h_1$), one finds the reinforcement height ($\Delta h_2$). Figure 4.1 indicate one simulation wherein the levee height $h_2$.

Per additional step height at $t=0$, all simulations of the Monte-Carlo method will be used. The investments and benefits will be calculated and summed up as described in equation 3.1.

Theoretically the expected value of the reinforcement height can be found by:

$$E[\Delta h_2] = \int_{-\infty}^{\infty} \Delta h_2 f(\Delta h_2) d\Delta h_2$$

(4.3)

Where:
- $E[\Delta h_2]$ Expected value of the reinforcement height [m]
- $f(\Delta h_2)$ Probability density function of the reinforcement height [-]

Unfortunately the probability density function of the reinforcement height is unknown. However, by simulating the reinforcement height with a Monte-Carlo simulation, one is still able to estimate its expected value:

$$E[\Delta h_2] \approx \frac{1}{n} \sum_{i=1}^{n} \Delta h_{2,i}$$

(4.4)

Where:
- $E[\Delta h_2]$ Expected value of the reinforcement height [m]
- $n$ Number of Monte-Carlo simulations [-]
- $\Delta h_{2,i}$ Reinforcement height as stochastic variable [m]

Hence, one searches the optimum NPV by varying the levee height at $t = 0$ and including the reinforcement height probabilistically given the uncertainties in hydraulic boundary conditions.

4.1.1. Benefits

The benefits, theoretically introduced in equation 3.7, depending on the flood risks of the present and new situation(s). Moreover, the risk depends on the levee height which varies during the optimization procedure. As this variable may differ over the analysis period, the benefits are separated in two different period. Equations 4.5 and 4.6 show this dependency.

$$B_1 = R_0 - R_1(\Delta h_1)$$

(4.5)

$$B_2 = R_0 - R_2(\Delta h_2) \quad \rightarrow \quad B_2 \approx R_0 - \frac{1}{n} \sum_{i=1}^{n} R_{2,i}(\Delta h_{2,i})$$

(4.6)
4. Cost effectiveness optimization

Where:

- $B$ Costs in the period between completion and major maintenance [€]
- $B_m$ Costs in the period between major maintenance and end-of-life [€]
- $R$ Flood risk of the present situation [€]
- $R_c$ Flood risk of the situation in the period between completion and major maintenance [€]
- $R_m$ Flood risk of the situation in the period between major maintenance and end-of-life [€]
- $R_{2,t}$ Flood risk simulation of the situation in the period between major maintenance and end-of-life from a Monte-Carlo simulation

- $\Delta h_1$ Additional levee height at $t=0$ [m]
- $\Delta h_2$ Reinforcement height at $t=T_m$ [m]
- $\Delta h_{2,t}$ Reinforcement height as stochastic variable at $t=T_m$ [m]

Substituting the summation of the Monte Carlo simulation (4.4) into the flood risk equation (3.6) lead us to the flood risks depending on the levee height ($\Delta h_1$) and the reinforcement height ($\Delta h_{2,t}$) found with the Monte-Carlo simulation:

$$R_0 = \int_{Q_0}^{Q_{\text{max}}} f(x) D(x) \, dx \quad \text{if } Q_0 > Q_{\text{max}} \rightarrow R_0 = 0$$

$$R_1 = \int_{Q_1(\Delta h_1)}^{Q_{\text{max}}} f(x) D(x) \, dx \quad \text{if } Q_1 > Q_{\text{max}} \rightarrow R_1 = 0$$

$$R_2 = \int_{Q_2(\Delta h_2)}^{Q_{\text{max}}} f(x) D(x) \, dx \rightarrow R_2 \approx \frac{1}{n} \sum_{t=1}^{n} \int_{Q_t(\Delta h_{2,t})}^{Q_{\text{max}}} f(x) D(x) \, dx \quad \text{if } Q_2 > Q_{\text{max}} \rightarrow R_2 = 0$$

Where:

- $Q_0$ Discharge corresponding with a probability of failure of the present situation [m$^3$/s]
- $Q_1$ Discharge corresponding with a probability of failure of the situation between completion and major maintenance [m$^3$/s]
- $Q_2$ Discharge corresponding with a probability of failure of the situation between major maintenance and the end-of-life time [m$^3$/s]
- $Q_{\text{max}}$ Maximum physical possible discharge of the river Meuse in Limburg [m$^3$/s]
- $f(x)$ Gumbel-max probability density function of the river Meuse discharge [-]
- $D(x)$ Expected flood damage [-]
- $n$ Number of Monte-Carlo simulations [-]

Theoretically the integrals goes to infinity. However, at a certain discharge ($Q_{\text{max}}$) many other levees will be flooded upstream the river Meuse. The probability that these types of floods cause damage in Arcen are much smaller as the Gumbel-max distribution suggests. The maximum possible discharge is set at 4.600 m$^3$/s which is in line with the current assumptions of the Dutch government (Berkhof et al. 2013).

The integral can be approximated with help of a Riemann sum:

$$\int_{Q_a}^{Q_{\text{max}}} f(x) D(x) \, dx \approx \sum_{f=Q_a}^{Q_{\text{max}}} f(Q_f) D(Q_f) \Delta Q$$

Where:

- $Q_a$ Discharge related to a certain probability of failure [m$^3$/s]
Combining equations 4.7 till 4.9 with equation 4.10, without discounting, leads to:

\[ R_0 \approx \sum_{j=0}^{Q_{\text{max}}} f(Q_j) D(Q_j) \Delta Q \quad \text{if } Q_0 > Q_{\text{max}} \rightarrow R_0 = 0 \]  
(4.11)

\[ R_1 \approx \sum_{j=Q_1(\Delta h_1)}^{Q_{\text{max}}} f(Q_j) D(Q_j) \Delta Q \quad \text{if } Q_1 > Q_{\text{max}} \rightarrow R_1 = 0 \]  
(4.12)

\[ R_2 \approx \frac{1}{n} \sum_{t=1}^{n} \sum_{j=Q_2(\Delta h_{2,t})}^{Q_{\text{max}}} f(Q_j) D(Q_j) \Delta Q \quad \text{if } Q_2 > Q_{\text{max}} \rightarrow R_2 = 0 \]  
(4.13)

Where:
- \( R_0 \): Flood risk of the present situation [€]
- \( R_1 \): Flood risk of the situation in the period between completion and major maintenance [€]
- \( R_2 \): Flood risk of the situation in the period between major maintenance and end-of-life [€]
- \( \Delta h_1 \): Additional levee height at Δh=0 [m]
- \( \Delta h_2 \): Reinforcement height at \( t=t_m \) [m]
- \( \Delta h_{2,t} \): Reinforcement height as stochastic variable [m]
- \( \Delta Q \): Riemann sum step size [m³/s]

### 4.1.2. Investments

The investments are introduced in paragraph 3.2 and depend on the variable and probabilistic levee heights \( \Delta h_1 \) and \( \Delta h_2 \). The project investments can be divided into constant investments (independent of the levee height) and variable investments (dependent on the levee height) as expressed in equation 4.14. Here the variable investment \( I_{pr,v} \) does not have to be linear. For instance, in order to maintain strength, some levee elements have to be widened when the variable levee height \( \Delta h_1 \) increases. So, an exponential or quadratic relations are thinkable.

\[ I_{pr} = I_{pr,c} + I_{pr,v}(\Delta h_1) \]  
(4.14)

Where:
- \( I_{pr} \): Project investment [€]
- \( I_{pr,c} \): Constant project investment [€]
- \( I_{pr,v} \): Variable project investment [€]

Major maintenance investments are separated into a constant and variable part as well. Furthermore, there is a preventive investment part which include maintenance costs during all circumstances at the execution time \( t = T_m \). There are possibilities that a levee does not have to be reinforced but do require some maintenance (i.e. renewing rubble connections). Including the dependency of the reinforcement height \( \Delta h_2 \) into the investments, without discounting, leads us to the following:

\[ I_{mm} = \begin{cases} 
  I_{mm,p} + I_{mm,c} + I_{mm,v}(\Delta h_2) & \text{if } \Delta h_2 > 0 \\
  I_{mm,p} & \text{if } \Delta h_2 \leq 0
\end{cases} \]  
(4.15)

\[ E[I_{mm}] = \begin{cases} 
  I_{mm,p} + I_{mm,c} + \frac{1}{n} \sum_{t=1}^{n} I_{mm,v}(\Delta h_{2,t}) & \text{if } \Delta h_{2,t} > 0 \\
  I_{mm,p} & \text{if } \Delta h_{2,t} \leq 0
\end{cases} \]  
(4.16)

Where:
- \( I_{mm} \): Major maintenance investment [€]
- \( I_{mm,p} \): Preventive major maintenance investment [€]
- \( I_{mm,c} \): Constant major maintenance investment [€]
- \( I_{mm,v} \): Variable major maintenance investment [€]
4.2. Hydraulic boundary condition uncertainties

Uncertainties in hydraulic boundary conditions for a river basically depends on four different aspects:

- The uncertainty in future river widening projects;
- the uncertainty in statistics;
- the uncertainty in forecasted climate changes;
- the uncertainty in data due to future floods.

These uncertainties are considered separately but do have a certain dependency. If an expected increase in river discharge due to climate change is less than thought, it probably affect the political decisions whether or not a river widening project will be executed. Within this thesis, these parameters are assumed to be independent. The variation in the resulting distribution of the hydraulic boundary condition uncertainties will probably be overestimated. Although, the variation is always equal or larger than the uncertainties in statistics.

Combining uncertainties should be carried out carefully. All aspects require the same reference parameters before adding them up together. The chosen reference parameters are the deterministically determined increase of the discharge in line with the Dutch government. Equation 4.17 shows the summation of all uncertainties including a deterministic increase ($\Delta h_{2,d}$). The deterministic scenario relates the well known Meuse discharge of 4.600 m$^3$/s with an annual probability of occurrence 1/1250 in the year 2100.

$$\Delta h_2 = \Delta h_{2,d} + \Delta h_{2,t} + \Delta h_{2,ii} + \Delta h_{2,iii} + \Delta h_{2,iv}$$  (4.17)

Where:
- $\Delta h_{2,d}$: Deterministic reinforcement height [m]
- $\Delta h_{2,t}$: Reinforcement height difference due to uncertainties in river widening projects [m]
- $\Delta h_{2,ii}$: Reinforcement height difference due to uncertainties in statistics [m]
- $\Delta h_{2,iii}$: Reinforcement height difference due to uncertainties in climate changes [m]
- $\Delta h_{2,iv}$: Reinforcement height difference due to uncertainties in future floods [m]

The different aspects are described in appendix D and summarized in the paragraphs 4.2.1 to 4.2.4. Additionally the uncertainties are included into the LCCA via a Monte-Carlo simulation. A sensitivity analysis has been made with a FORM-analysis in paragraphs 4.2.6.

4.2.1. Uncertainty in future river widening projects

The Dutch government currently develops long term plans to balance the prospected increase in the dispersion of extreme weather conditions. These plans are in line with the preferred strategy: "River widening measures when possible, reinforcing levees when needed". As a result, several areas along the river Meuse in Limburg are assigned to become future river flood plains during extreme conditions. Together with many stakeholders maps have been developed to analyse possible future flood plain areas. Next, two strategies are formed including model calculations. The outcome is an increase in water level with one of these strategies (i.e. so included the river widening project which decreases the water levels). According to the report Berkhof et al. (2013), the outcome is only indicative. Nevertheless, it gives some basic understanding to the influence of possible future river widening projects.

Unfortunately, only two strategies are shown, one with no extra river widening projects, and the other one where most assigned areas become river widening projects. Using those two strategies as minimum and maximum scenario’s with a 10% confidential interval (5% at each side) parameters of distribution can be found. Furthermore a triangular distribution is chosen which entails relatively large chance that the actual increase in water level is somewhere in the middle. Table 4.1 shows all the resulting parameters of the triangular distribution. The current river widening prospects (without any river widening project) is extracted of the values such that the zero reflects the situation related to a 4.600 m$^3$/s discharge in the year 2100. The 0,64 meter resembles the deterministically reinforcement height ($\Delta h_{2,d}$).
4.2. Hydraulic boundary condition uncertainties

4.2.2. Uncertainty in statistics

Discharges of the river Meuse are measured since the year 1911. Over the years, the technique to measure discharges differs, influencing the total outcome. Secondly, due to the short period of data in relation to the probabilities of governing floods (1/250 to 1/1250), there is a large extrapolation error. Thirdly, there is no exact distribution which translates the exact data 100% accurate. Some are close like the Gumbel-, Pearson III- or the Exponential-distribution, but still differ significantly. Together with the data available, a normal distribution is chosen to cope with the error in discharge due to measure-, extrapolation- and distribution type-differences.

A normal distribution with a mean value of zero and a standard deviation of 0,15 meter is assumed for the uncertainties in statistics.

4.2.3. Uncertainty in forecasted climate changes

Forecasted climate changes affect the river Meuse discharges. Currently the Dutch government uses forecasting scenario’s drawn by the KNMI. The variation within these scenario’s is used to reflect them with a rain run-off model and rules of thumb. On average, a temperature increase of 1% Celsius lead to an approximately 5% discharge increase according to literature (de Wit 2004, Jansen 2007). The rain run-off models logically provide a more scientific forecast. Unfortunately however, the outcome does not coincide with the required information. Therefore, the distribution and parameters are based on KNMI scenario’s of 2006. A uniform distribution is chosen with the outer two scenarios as upper and lower bound with a 5% confidential interval. Again, the prospected increase from 4.000 (today’s 1/1250 prospected discharge) to 4.600 $m^3/\text{s}$ in the year 2100 is used as zero value. So, if the discharge increases exactly as defined in this reference scenario, which is used by the Dutch government, there is no difference to the prospected discharge c.q. water level increase due to climate change.

Hence, a uniform distribution is assumed with a lower bound of -0,28 meter and an upper bound of 0,13 meter.

4.2.4. Uncertainty in data due to future floods

A research to actual uncertainty due to future possible floods, by altering historical data, has to be carried out including the dependence of different distribution types. Such investigation has been carried out but not fully applicable to the whole analysis of uncertainties in hydraulic boundary conditions. As the additional uncertainties due to future possible flood probably is relatively small in relation to the other uncertainties, and because it is expected that they contribute within the uncertainties in forecasted discharge, they are not taken into account within this research. The available data is used to describe as accurate as possible future potential floods (i.e discharges related to flood probabilities).

4.2.5. Monte-Carlo simulation results of the hydraulic boundary condition uncertainties

The uncertainties in hydraulic boundary conditions mentioned above are simulated with a Monte-Carlo-model. Figure 4.2 shows the results of this simulation. The relatively large spread of the expected levee reinforcement $h_{2,\text{r}}$ is clearly visible.

The expected value of the levee reinforcement height in the year 2070 ($h_{2,\text{r}}$), to fulfill the required safety level in 2120, is calculated from data of the Monte-Carlo simulation:

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4.2.6. Sensitivity analysis

A sensitivity analysis has been carried out with a FORM-simulation. This simulation requires a failure function which is defined as follow:

\[ Z \leq 0 \]  
\[ Z = \Delta h_{2,d} + \Delta h_{2,f_j} + \Delta h_{2,f_l,j} + \Delta h_{2,f_l,j+l} \]  

The influence coefficients are presented in table 4.2. It appears that all parameters contribute significantly to the total hydraulic boundary condition uncertainties.

<table>
<thead>
<tr>
<th>Uncertainty</th>
<th>Influence factor</th>
<th>as percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Forecasted discharges</td>
<td>0,5131</td>
<td>31%</td>
</tr>
<tr>
<td>Future climate changes</td>
<td>0,7774</td>
<td>47%</td>
</tr>
<tr>
<td>Future river widening projects</td>
<td>0,3638</td>
<td>22%</td>
</tr>
</tbody>
</table>

The sensitivity analysis is performed only for the hydraulic boundary conditions. Other uncertainties like costs, discount rate, growth rate and benefits are not determined probabilistically, and are therefore not included into the FORM analysis.

4.3. Results

Each realistic alternative from chapter 2 is analysed with the optimization procedure. Figure 4.3 illustrates the results of one alternative. It turns out that all alternatives in both sections have the same optimization height of 0,6 meter (with a step size of 0,10 meter).
4.3. Results

Figure 4.3: Benefits, Investments and Net Present Value of alternative C, section 1: case study Arcen

Table 4.3: Deterministic determined investments of alternative C, section 1: case study Arcen with and without discounting (no robustness)

<table>
<thead>
<tr>
<th>Investments</th>
<th>Without discounting [€/m]</th>
<th>With discounting [€/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>4.661,81</td>
<td>4.661,81</td>
</tr>
<tr>
<td>Service (t=1)</td>
<td>269,67</td>
<td>113,90</td>
</tr>
<tr>
<td>Service (t=10)</td>
<td>12,15</td>
<td>4,94</td>
</tr>
<tr>
<td>Reactive maintenance</td>
<td>0,00</td>
<td>0,00</td>
</tr>
<tr>
<td>Major maintenance</td>
<td>1.019,22</td>
<td>360,56</td>
</tr>
<tr>
<td>Demolish costs</td>
<td>496,82</td>
<td>62,16</td>
</tr>
<tr>
<td>Total</td>
<td>6.459,67</td>
<td>5203,39</td>
</tr>
</tbody>
</table>

Figure 4.4 shows all deterministically formed present value investments per realistic levee design alternative and per aspect. The sum of the aspects reflecting the total LCC. A rate is used of 2,1% consisting of the discount rate and the economical growth rate.

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The NPV has been calculated by three different methods:

- Deterministically;
- deterministically including the resulting optimization of 0.6 meter;
- probabilistically (with the optimization height).

Table 4.4 summarizes the total NPV calculation for the whole ring-levee for all three methods per levee design alternative. A high NPV result in a cost-effective levee design. A sequence is given per section reflecting the economical preferential at the right hand side of each result. The largest NPV is stated first, the second largest second, and so forth.

<table>
<thead>
<tr>
<th>Alternative</th>
<th>NPV Methods</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Det. $\times 10^6$</td>
</tr>
<tr>
<td>1 C</td>
<td>€ 55,84 (2)</td>
</tr>
<tr>
<td>1 D</td>
<td>€ 55,90 (1)</td>
</tr>
<tr>
<td>1 K</td>
<td>€ 55,81 (3)</td>
</tr>
<tr>
<td>2 B</td>
<td>€ 57,46 (1)</td>
</tr>
<tr>
<td>2 C</td>
<td>€ 56,10 (3)</td>
</tr>
<tr>
<td>2 E</td>
<td>€ 57,12 (2)</td>
</tr>
<tr>
<td>2 K</td>
<td>€ 55,53 (4)</td>
</tr>
</tbody>
</table>

A Matlab script is used to calculate all three methods, although it is not necessarily the most affective method to form the deterministic calculations. Via this manner, all parameters are kept the same. Interpolations of data could slightly differ which decreases the accuracy of the outcome. Figure 4.5 shows the histogram of each levee design alternative per method.

![Figure 4.5: Column graph showing results from table 4.4](image)
4.4. Preliminary conclusion

This paragraph states a preliminary conclusion for case study Arcen. Results of the probabilistic calculations with the optimization procedure are analysed with respect to the deterministic calculations with and without the optimization height. Chapter 5 validate the elaborated optimization procedure and this conclusion with case study Venlo.

The benefits per alternative do not differ much due to two reasons: There is no significant change in levee alignment and all levee design alternatives have the same optimal levee height with the resulted robustness ($\Delta h_1$).

The effect of the maximum physically defined discharge is visible in the graph of figure 4.3. At the robustness of 0,7 meter, the benefits becomes horizontal. So, if one include a robustness of 0,7 meter or more, the related discharge becomes larger than the assigned maximum discharge ($Q_{max}$).

The investments (figure 4.1) increase over the extra robustness. The conclusion can be drawn that, extra robustness in order to decrease the probability of failure due to overflow/overtopping, costs more when executing directly. Notice the investments are determined entailing the whole ring-levee, not the particular sections of interest. Benefits of flood risk reduction due to heightening of the levee only increase if the whole ring-levee is increased.

The project investments dominates by far the total investments if the described discount- and economical growth rate of 2,1% is applied. No direct conclusion can be drawn that a basic cost estimate without using LCCA does not encompasses the required outcome.

Both rates are economical and political estimations. A difference in these rates could encompass significant changes in the alternative analysis outcome, and subsequently in the relative influence of the investment aspects besides the project investment. Figure refig:importance-discountovoet illustrate the total investments per realistic levee design alternative with respect to the discount rate minus the economical growth rate ($r - g$). There are no intersections between levee design alternatives. Hence,

![Image of Figure 4.6](image.png)

Figure 4.6: Graph showing the effect of the discount- and growth rate to the deterministically determined present value of the investments at case study Arcen.

a change in the total rate does not affect the sequence of the alternatives. Although some alternative
investments do converge or diverge.

The differences in NPV calculations are the actual differences of the sections only. The reason the whole ring-levee is included was to come up with the optimal levee height including the extra robustness ($\Delta h_1$). As all alternatives have the same robustness, all other parts of the levee have the same NPV (The benefits differ due to a variable outcome of the Monte-Carlo simulation).

The sequences per section do not differ much. Only two alternative changes in the Probability method with respect to the other two methods 2C and 2E (table 4.4). One is able to divide the difference between the deterministic (Det. + opt.) and probabilist method (Prob. + opt.) by the probabilistic method per alternative:

$$\text{relative difference} = \frac{NPV_{\text{Prob.}+\text{opt.}} - NPV_{\text{Det.}+\text{opt.}}}{NPV_{\text{Prob.}+\text{opt.}}}$$

(4.21)

If the difference between the alternatives is large, the inclusion of the hydraulic boundary condition combined with the optimization procedure changes the outcome. Figure 4.7 illustrate the relative differences per alternative in a pie-diagram.

![Pie Chart](image)

Figure 4.7: Distribution of the differences between the deterministic optimization (including the 0.6 meter robustness) and the probabilistic calculation. 100% represent the sum of the relative differences of all alternatives

There is no significant difference between the alternatives outcome of the deterministic with robustness method and the probabilistic method. Hence, if the optimal levee height is known, there is no need to include the hydraulic boundary condition uncertainties into the levee design alternative analysis within case study Arcen.

It seems that, based upon the calculations carried out, and given the assumptions made, it is economically attractive to implement the new safety philosophy directly. Even though the investments are larger, it reduces flood risks resulting in much less flood damage within the analysis period.

Table 4.4 shows an approximate 10 million euro difference between the deterministic method without robustness and the other two methods. Thus, with the assumptions made, one has approximate 10 million euro of present value saved due to reinforcing with extra robustness. To elaborate these savings into perspective, dividing them by the analysis period (100 years) and ring-levee length (5000 meter) lead us to:

$$\Delta NPV = \frac{NPV_{\text{optimum}} - NPV_{\text{no optimum}}}{T \times L_{\text{tot}}} \approx \frac{10 \times 10^6}{100 \times 5000} = 20 \text{ [€/m/y]}$$

(4.22)
4.5. Discussion

The preliminary conclusion elaborate two aspects: The robustness and the alternative analysis. As there is no difference in robustness over the alternatives, one is able to split these aspects into two calculations. One determines the total robustness of the ring-levee while the second one uses LCCA to investigate the economical differences between levee design alternatives. Results of this research show that hydraulic boundary conditions uncertainties are only important in the first calculation.

Assumptions related to the probability of failure are very rough. Overflow or overtopping does not necessarily cause the same economical damage as related to the maximum inundation height. Moreover the required probability of failure related to the new safety philosophy can be much different than the used annual probability of 1/1250 used here. However, the NPV increases and is positive between 0 and 0,6 meters. So, it is wise to design a levee based on the new safety philosophy, too.

In this research, if a levee design alternative is situated at the river-side of the present levee alignment, the reduced area of the river flow profile has to be compensated. The costs of this compensation are included within the LCCA. Of course the actual impact has to be investigated later on. Moreover the compensation is an other project at its own, entailing other stakeholders and additional requirements. But this does not imply to cancel out all alternatives with a levee alignment to the river side beforehand.

Even with uncertainties due to several assumptions, the benefits are by far governing within the analysis. Hence, calculating the benefits differently result in an other outcome, but it is very unlikely they result in a different conclusion regarding the analysis between alternatives.

The physical maximum discharge of the river Meuse in Limburg is assessed to be $4.600 \text{ m}^3/\text{s}$ (van den Berg et al. 2013). Within this research this maximum discharge is applied over the whole analysis period of 100 years. However, future national and international developments of the Meuse region could lead to a different maximum. If it is less than the stated value above, it will affect the robustness outcome enormously as it drops down almost equal with the maximum water height. If the maximum discharge turns out to be more, the robustness could enlarge. The calculations carried out here should be repeated with the new physical maximum discharge in order to find the new optimum robustness levee height.

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Case study Venlo

In this chapter a second case study is set out which is used to “validate” the preliminary conclusions of the alternative analysis carried out in chapter 4. Previously, realistic alternatives are formed in chapter 2 and analysed in the subsequent two chapters for case study Arcen. This case study Venlo uses the formed optimization procedure during case study Arcen in order to analyse the preliminary conclusion stated.

Theoretically, it is hardly possible to validate the preliminary conclusions properly. A large number of cases should be investigated in order to have a accurate reliable conclusion. The first case study is used during the research. If results seems to fit the expectations, it does not necessarily mean the conclusion related to the results is 100% correct for all urban areal cases along the river Meuse in Limburg. The case study of Venlo will be analysed with the same procedure as case study Arcen, but without further mathematical adjustments to the procedure.

First case study Venlo is introduced followed by the NPV calculation in paragraph 5.3 which differs from the other case study. Next the hydraulic boundary conditions are formed in paragraph 5.4 followed by the results of case study Venlo and the validation in paragraph 5.6.

5.1. Introduction to case study Venlo

Venlo is an old city at the Eastern part of the river Meuse in Limburg. Together with the village Blerick, which is located at the western side of the Meuse, and other villages they form the municipality of Venlo.

The cities economy is mainly orientated to the trade-, transport- and the industrial market, partly due to the appropriate location nearby the German border. Although the city has been bombed during the second world war, some medieval buildings are still present in the city centre (e.g. the Romerhuis).

Along the river Meuse in Venlo, a relative new boulevard is constructed. The levee structure partly consists of a demountable stop-log barrier which does not fulfil the present safety requirements. Secondly, Serverice costs of this type of barrier are high compared with permanent levee types. Besides stocking of the material, one has to test the flood defence. At several cases in Venlo a new design is wanted with consideration of the new safety philosophy. An example of the present situation is shown in figure 5.1.

Currently a new project along the flood defence is in progress (both planning phase as construction phase), named Q4. An impression sketch is shown in figure 5.2. Old premises are demolished to construct new one's with a nice view to the river Meuse. One of the primary preference regarding the flood defence from the municipality of Venlo is to preserve this view as much as possible.
5.2. Defining, designing and screening alternatives

Alternatives are defined, designed and screened for case study Venlo with help of the toolbox of appendix C. Figure 5.3 shows all realistic design alternatives of the case study.

5.3. NPV calculation

The calculation procedure of both case studies will be similar as much as possible. However, there are some extra aspects regarding the NPV calculation of case study Venlo. These aspects are stated here.

Unfortunately, it is a difficult task to assess the probability of failure of the whole ring-levee. An optimum is found including an extra height robustness by increasing steps over the entire ring-levee. In this case study, the annual probability of failure per levee section varies between 1/70 and the 1/1,500 (VNK 2014). Furthermore, the levee consist of many different and small levee elements.

Some assumptions are made in order to achieve enough data to perform the optimization calculation. The assumptions are based on the draft version of the report of VNK (2014). In short, only the Southern part of the ring-levee is assessed with an approximated length of 7,500 meter. The economical damage of these parts are assessed separately within the report of VNK (2014), too. Within this model two sections will be assessed: Section 3 and 4. All other levees in the Southern part are clustered together in two groups: Green and structural levee types. Each element will be reinforced in the years 2020 and/or 2070 depending on the present annual probability of failure. Subsequently,
5.3. NPV calculation

(a) Case study Venlo: Realistic alternative 3B

(b) Case study Venlo: Realistic alternative 3E

(c) Case study Venlo: Realistic alternative 3F

(d) Case study Venlo: Realistic alternative 3L

(e) Case study Venlo: Realistic alternative 4B

(f) Case study Venlo: Realistic alternative 4C

(g) Case study Venlo: Realistic alternative 4D

(h) Case study Venlo: Realistic alternative 4FL

(i) Case study Venlo: Realistic alternative 4I

Figure 5.3: Realistic alternatives case study Venlo

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the average failure probability is calculated with respect to the levee element lengths. Thus, instead of using a vast amount of levee sections, one uses 4 groups. Their relative importance are averaged via their length. A more extensive elaboration is given in appendix F.

In this research, all levees in Venlo have to fulfil the present statutory assessment with an annual exceedance probability of 1/250 in the year 2020. Subsequently, all levees should fulfil the requirements of the new safety approach with an annual failure probability of 1/1.1250 in the year 2070 (Berkhof et al. 2013). Hence, some levee elements will be reinforced in the year 2020 and, if necessary, in 2070. Other levees fulfil the present statutory assessment and will be reinforced in the year 2070.

5.4. Hydraulic boundary condition uncertainties
The uncertainties in hydraulic boundary conditions for case study Venlo are formed and shown in figure 5.4. The main difference between the case studies Arcen en Venlo can be found in the river widening projects. The statistics and climate change are based on discharges and assumed to be equal for both case studies. However, the Q-h relation of each case differs. Furthermore the size of the high water wave is different. The expected increase of the water level related to the annual probability of failure of 1/1250 and the discharge of 4.600 \(m^3/s\) in Venlo is 1,01 meter. The mean value of the major maintenance increase resulting from the Monte-Carlo simulation is:

\[
\Delta h_2 = 0,7237 \text{ meter} \tag{5.1}
\]

5.5. Results
Each realistic alternative from paragraph F.5 is analysed with the optimization procedure. Figure 5.5 illustrate the total benefits, investments and NPV of the whole ring levee for alternative F section 3. All alternatives have the same optimization height of 0,8 meter (with a step size of 0,10 meter). Table 5.1 shows the investments per aspect of this alternative with and without accounting the discount- and economical growth rate.

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5.5. Results

Table 5.1: Deterministic determined investments of alternative F, section 3: case study Venlo with and without discounting (no robustness)

<table>
<thead>
<tr>
<th>Investments</th>
<th>Without discounting [€/m]</th>
<th>With discounting [€/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project (t=1)</td>
<td>2.675,95</td>
<td>2.675,95</td>
</tr>
<tr>
<td>Service (t=1)</td>
<td>372,99</td>
<td>157,54</td>
</tr>
<tr>
<td>Service (t=10)</td>
<td>588,00</td>
<td>239,26</td>
</tr>
<tr>
<td>Reactive maintenance</td>
<td>16,14</td>
<td>6,70</td>
</tr>
<tr>
<td>Major maintenance</td>
<td>2,708,45</td>
<td>958,15</td>
</tr>
<tr>
<td>Demolish costs</td>
<td>92,89</td>
<td>11,62</td>
</tr>
<tr>
<td>Total</td>
<td>6.454,42</td>
<td>4.049,22</td>
</tr>
</tbody>
</table>

Figure 5.6 shows all deterministically calculated present values of the investments per realistic levee design alternative and per aspect. The sum of the aspects reflecting the total LCC. A rate is used of 2.1% consisting of the discount rate and the economical growth rate.

The NPV has been calculated for the same three different methods: Deterministically, deterministically including the resulting optimization of 0.8 meter and probabilistically (with the optimization height).

Table 4.4 summarizes the total NPV calculation for the whole ring-levee for all three methods per levee design alternative. A high NPV result in a cost-effective levee design. A sequence of the NPV is given per section at the right hand side of each result.

Figure 5.7 shows the histogram of each levee design alternative per method. It is clearly visible that there is not much difference between the deterministic and probabilistic method both including robustness.

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### Figure 5.6: Overview of the deterministically formed present value of the investments per realistic levee design alternative of case study Venlo.

![Figure 5.6: Overview of the deterministically formed present value of the investments per realistic levee design alternative of case study Venlo.](image)

Table 5.2: Results of three NPV calculations: Deterministically (Det.), deterministically including a 0,8 meter robustness (Det.+Opt.) and Probabilistic including a 0,8 meter robustness (Prob.+opt.) for case study Venlo

<table>
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<tr>
<th>Alternative</th>
<th>NPV Methods</th>
<th>Det. (\times 10^6)</th>
<th>Det. + opt. (\times 10^6)</th>
<th>Prob.+opt. (\times 10^6)</th>
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<tr>
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<td>€ 31,00 (1)</td>
<td>€ 31,36 (1)</td>
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<td>3 E</td>
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<td>€ 30,98 (2)</td>
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### 5.6. Conclusion of case study Venlo

The conclusion of case study Venlo can justify or contrary disapprove the preliminary conclusion of case study Arcen. Notice that just one additional case study is used during the validation.

One preliminary conclusion of case study Arcen refers to the effect of the maximum physical possible discharge of the river Meuse. This maximum is clearly visible in case study Venlo as can be seen in figure 5.5. The benefits at the robustness height of 0,8 meter become abruptly constant. In fact, the optimum NPV has the same levee height (\(h_1\)) as the threshold of the benefits at 0,80 meter. Hence, the influence of the physical maximum discharge is even more significant in case study Venlo.

The investments of all realistic levee design alternatives in case study Venlo increasing over the extra robustness which coincide with the preliminary conclusion drawn for case study Arcen.

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In contrast to the preliminary conclusion there is no full dominance of the project investments. Other aspects like major maintenance and service investments contribute significant, too. Using a basic cost estimate with only project investments lead to an insufficient outcome. Secondly, the sequence differs over the alternatives (e.g. 4F and 4I) and the total present values are converged much closer (e.g. 3B and 3E) with LCCA.

The influence of the discount- and economical growth rate is presented in figure 5.8. There are several intersection between the alternatives, especially around the 2%. Hence, a change in the total rate does affect the sequence of the alternatives.
Alternatives 3B and 4B have the same design option types as the present situation. Although they are attractive by all NPV calculations with the stated 2,1% rate, they become less preferable with a lower rate.

The sequence per section does not vary between the three methods. Again equation 4.21 is used to investigate the relative difference between the probabilistic and deterministic methods, both with the robustness. The relative difference varies between the 11 an 12%.

For case study Venlo, it seems that, based upon the calculations carried out, and given the assumptions made, it is economically attractive to implement the new safety philosophy directly. In contrast to applying the present safety standard only, one obtain relative more benefits for their levee investments. Moreover the outcome encompasses that it is economically unattractive to execute measures without any robustness. The NPV is negative (both in the probabilistic and deterministic method) at $\Delta h_1 = 0$. Notice the average of the present safety level is used (annual probability of exceedance of 1/178) rather than considering all sections within the ring-levee separately. A section with a high probability of exceedance might still be beneficial to increase without extra robustness. This way of dealing with investments and flood risk reductions is in line with the new safety philosophy. Remember the VNK-approach wherein one calculates per measure the risk reduction.

Hence, although the methodology of considering limited sections and averaging the other parts of a ring-levee seems to work for the Arcen case, it does not lead to a conclusive answer.

The robustness of 0,8 meter results in an expected increase in benefits of approximate 30 million Euro present value, leading to twice as much as the sections in case study Arcen:

$$\Delta NPV = \frac{NPV_{optimum} - NPV_{no\ optimum}}{T \cdot L_{tot}} \approx \frac{30 \cdot 10^6}{100 \cdot 7500} = 40 [\text{€}/\text{m}/\text{y}]$$  \hfill (5.2)
Conclusion and recommendation

The objective of this MSc-thesis is to investigate which balance between levee investments and robustness is most cost-effective during the levee design alternative analysis, specified for cases along the river Meuse in Limburg. Within this research LCCA is used which includes the benefits of a flood defence as well. An optimization model is used which includes the uncertainties in hydraulic boundary conditions probabilistically. Given the present safety requirements and the requirements related to the new safety philosophy, one is able to find the optimal robustness height of a levee design. Two case studies are investigated wherein multiple levee design alternatives are assessed with the optimization procedure.

Chapter 2 defined, designed and screened the levee design alternatives of the case study Arcen. The realistic alternatives are investigated further within the research. A theoretical framework of the LCCA method has been stated in chapter 3. Next an optimization procedure is formed where the optimal robustness of the levee height is found per realistic alternative for case study Arcen in chapter 4. Within this optimization procedure hydraulic boundary condition uncertainties are included probabilistically. The chapter ends with a preliminary conclusion. Subsequently, chapter 5 introduced case study Venlo which has been used to evaluate the previous mentioned conclusion. This chapter states the final conclusion and recommendation of the MSc. thesis research.

6.1. Conclusion

Key-questions are formulated in chapter 1 to answer the main research question: What are cost-effective levee designs including uncertainties in hydraulic boundary conditions?

How can life-cycle costs of different levee types and alignments be compared equally?

Some levee types enhance difficult demountable or dynamic systems, while others consist mostly of earthy material. LCCA is an appropriate technique to compare them with the knowledge of many uncertainties. It includes among others project investments, major maintenance costs, service costs and benefits. Shortly, the benefits of a levee investment can be found by calculating the flood risk reduction.

Although the contribution of the LCC aspects besides project investments, like service costs or maintenance costs, do not affect the alternative LCCA outcome, they do change the relative importance between the levee design alternatives. Focussing only on the project investments leads to an error of approximately 30%.

By including benefits one is able to investigate the minimum required robustness in terms of levee height in order to obtain an economical attractive levee design alternative. Besides it create a honest comparison between levee design alternative with different alignments, it shows whenever an alternative is economical attractive.
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6. Conclusion and recommendation

All Investments are included within the LCCA with the implementation of a discount rate. This real discount rate of 2,5% is embedded by the Dutch government. Furthermore an economical growth rate is assumed of 0,4%. However, no specified economical growth data is available for levee design purposes. Both rates combined result in a total rate of 2,1%. This rate is varied within the LCCA in both case studies to investigate its dependency. Due to the fact that these rates are forecasts, they are uncertain. If these rates turn out to be different, it could affect the economical preferences of the levee design alternatives. The combined rate is investigated on its sensitivity for a deterministic calculation. In both cases the influence becomes very small for rates higher than 2,1 a 2,5%. Literature introduces a possible additional discount rate of 3% to include future uncertainties. Hereby one concludes that: Adding a 3% additional rate to the 2,5% real discount rate, does not have any effect for both case studies. In contrast, a smaller rate than 2,1% has a significant influence.

Thus, LCCA is an appropriate tool to investigate various types of levees as it includes project costs, maintenance costs, service costs and End of lifetime costs. They can only be compared equally if benefits are included as well. Furthermore the discount rate and economical growth rate are of significant importance.

How large are the uncertainties in hydraulic boundary conditions?

Three different aspects are analysed quantitatively regarding the uncertainties in hydraulic boundary conditions:

- The uncertainty in future river widening projects;
- the uncertainty in statistics;
- the uncertainty in forecasted climate changes.

A future flood may change the current data set resulting in a change in parameters c.q. distribution of the present statistics. The current uncertainties in statistics are significant. These uncertainties may include the uncertainty due to future floods as well.

The outcome of the hydraulic boundary condition uncertainties are plotted in figures 6.1 and 6.2 for both case studies. They show the required levee increase during major maintenance in the year 2070 if these levees fulfil the present statutory assessment in the year 2020. \( \Delta h_2 \) is related to the probabilistically determined water levels in the year 2120 with the annual failure probability of 1/1250. Mostly due to a different Q-h relation and the effect of possible river widening projects are causing the disparity between the case studies outcome.

![Histogram of Hydraulic boundary condition uncertainties](image)

Figure 6.1: Monte Carlo simulation: Combination hydraulic boundary condition uncertainties of river widening projects, statistic and climate changes. The simulation is specified for case study Arcen and represents the dispersion of \( \Delta h_2 \) required to fulfill the annual probability of failure of 1/1250.

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The sensitivity of the three aspects to the hydraulic boundary condition uncertainties is investigated. The uncertainties in statistics, climate change forecasts and future river widening projects are all governing components according to the outcome of the FORM-analysis.

Although the uncertainties in hydraulic boundary conditions are formed with rules of thumb and no comprehensive model calculations, they give a good insight in the total dispersion and scale. One can see clearly that, it is most probable that the levees have to be reinforced, or large river widening projects have to be executed in order to satisfy the new safety philosophy requirements if the levees are increased up to the 1/250 safety level in accordance with the present statutory assessment.

**How can the cost-effectiveness be investigated with inclusion of the uncertainties in hydraulic boundary conditions?**

The main function of a levee is to reduce flood risk. A higher levee design results in a larger flood risk reduction regarding the failure mechanism overflow and overtopping. One tries to account for future changes in hydraulic boundary conditions within the levee design. In some cases it is wise to reduce the flood risk further by investing more. This extra investments, or robustness, has its optimum. If future water level heights are less than expected, extra robustness becomes useless. Hence, the robustness is not beneficial during all circumstances.

Within this research an optimization tool is used in which the Net Present Value (NPV) of each realistic levee design alternative is optimized. The NPV can be found by subtracting the present value of the investments from the present value of the benefits. The new safety approach is assumed to become a requirement after 50 years. Thus, depending on the levee height of the current levee design, major maintenance has to be executed in order to fulfil the new safety standards. Furthermore the water level heights are changing due to several hydraulic aspects like climate change. The total levee height depends on these aspects, which are uncertain. Including extra robustness within the present levee design increases benefits and decreases major maintenance costs. The optimal robustness can be found by taking the argument of the maximum NPV within the optimization procedure as can be seen in equation 6.1. The total optimal levee height is a summation of this extra robustness and the minimal required levee height of the present safety approach.

\[ h_1 = h_{\text{min,1}} + \arg \max_{\Delta h_1} NPV \] (6.1)

Hence, the cost-effectiveness can be investigated by varying the robustness, calculating the NPV per step and taking the robustness related to the maximum NPV. Within these calculations, all NPV-aspects depend on future hydraulic boundary conditions which are included probabilistically. Within this research only failure due to overflow and overtopping are included. Other failure mechanisms do influence the cost-effectiveness, but the levee height is the significant parameter encompassing the most resistance from other stakeholders.

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What influence does the probabilistic method have on the levee design alternative analysis?

The preferred sequence of levee design alternatives, purely based on the NPV calculation, is formed for three different methods:

- Deterministically;
- deterministically including robustness;
- probabilistically including robustness.

Hydraulic boundary conditions influence the preferred sequence if there is a significant difference between the deterministic and probabilistic methods. However, this is not the case. The Results of case study Arcen clearly indicate that, although the absolute difference between the two methods over the alternatives are present, the relative difference between the probabilistic and deterministic method is 14 a 15% for all realistic levee design alternatives. This conclusion is confirmed with case study Venlo. There, all alternatives lay within the 11 and 12%\(^1\). Hence, even though there is a difference between the probabilistic and deterministic method (both with the robustness), these differences are relatively equal. In other words, the \( X \) in equation 6.2 for all alternatives is almost equal.

\[
\text{NPV}_{\text{Probabilistic method+robustness}} = X \times \text{NPV}_{\text{Deterministic method+robustness}}
\] (6.2)

Thus, the hydraulic boundary condition uncertainties do not contribute significant within this analysis on choices between levees regarding the financial aspect.

How large is the robustness for a cost-effective levee design?

Besides the investigation to the relative difference of the realistic alternative LCC, the significance of the robustness has been analysed. By varying the levee height, and including the uncertainties in hydraulic boundary conditions, one is able to find the optimum robustness at the highest NPV for each alternative. Both case studies have a large optimum robustness. An extra levee increase of 0,6 meter on top of the levee height related to the 1/250 safety level is economically most preferred for the Arcen case. For the case of Venlo a height of 0,8 meter is found as the optimum robustness.

The benefits have the highest impact into the NPV calculations partly due to the current large flood risks. Other parts of the Netherlands, in which the flood risk has been reduces over centuries, probably have lower benefits.

There is negligible difference in optimal robustness over the alternatives. The main logical reason behind the similar robustness height can be found in the size of the ring-levee compared to the size of the sections of interest. The effect of the levee design alternative investments in relation to the whole ring-levee is very limited. Although, the differences between the NPV's are caused by the levee section parts only.

Assumptions are made during the research encompassing essential differences with the actual case. Nevertheless, these results show that it is economically attractive to include robustness into the levee design. For case study Arcen it is beneficial to upgrade the levees to an annual probability of failure of 1/250 (The NPV is positive in all steps within the optimization), but even more attractive to include the requirements of the new safety philosophy directly. For the case of Venlo it is inefficient to upgrade its levee by the use of only the present safety approach (Negative NPV).

Many different levee types with various failure probabilities are present in case study Venlo leading to future upgrade of many small levee sections. The total method to design and assess levees with the new safety philosophy is still under development. In this philosophy, failure probabilities per section and mechanisms have to be gathered leading to a total annual probability of failure for a ring-levee

\(^1\)The difference between the case study Arcen (14 ñ 15%) and Venlo (11 ñ 12%) occurs due to the difference in the number of alternatives.
or levee-part. If the design methodology is equal as the VNK, it can be used perfectly to assess the levees along the river Meuse in Venlo. Due to the relative large robustness found in this research, a long-term plan is preferred for Venlo.

A maximum physically possible discharge \((Q_{\text{max}})\) has been defined of 4,600 \(m^3/s\) which is clearly visible in the benefits of the graphs at both case studies (figure 4.3 and 5.5. Hence, there is a large influence of this maximum discharge to the optimal robustness calculation.

6.2. Recommendation

The following recommendations are made regarding levee designs, hydraulic boundary conditions and LCCA in general:

- Use the new safety philosophy within the levee design stage;
- Give more input to the effect of future river widening projects. Currently only two scenario’s are published. Furthermore a relation can be created between future river widening projects and other hydraulic boundary condition uncertainties. For instance, these areas are becoming river widening projects if the water level with a certain probability raises up to a certain water level;
- Do not choose between levee design alternatives based on their project costs only, but use LCCA or another simular technique which include service- and maintenance costs as well;
- Do not include hydraulic boundary condition uncertainties within the levee design alternative analysis.

The following recommendation is related to case study Arcen:

- Especially within section 1 there are several appropriate levee design alternatives which have their alignment at the river side of the present situation. At the other side, an agricultural area can be lowered in height to maintain the water levels, neglecting the negative hydraulic impact of these alternatives. It is recommended to analyse these possibilities as well.

The following recommendations are related to case study Venlo:

- There are many demountable systems present along the river in and nearby Venlo. Many do not fulfil the present safety standards partly due to a lack in proper data. meanwhile several years have passed by "creating" additional possible data. This data is of interest to reinvestigate the present statutory assessment and/or investigate the failure within the new safety philosophy;
- the demountable systems are included in both sections as a possible alternative. Although they do not fulfil the present statutory assessment, with the assumptions made, this type of structure (with a relative large service costs) are economical attractive.

The following recommendations are made for future research:

- Investigating the actual physically possible maximum discharge of the river Meuse \((Q_{\text{max}})\) as it contribute significantly to the outcome of the optimal robustness height;
- investigate the real discount- and growth rate specified for levee design alternatives. The discount rate partly include the belief that one has more purchasing power by expending money in the future rather than today. However, governmental expenses do not work like that. Furthermore the conclusion set out their is a significant dependency of the rates related to the levee design alternative analysis.
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Design information case study Arcen

In order to research the cost-effectiveness of alternatives properly, a case study has been carried out. During the development of the optimization procedure this case study will use all values, figures and answers are actual results which can be used in the future for this case. This appendix contains data, calculations and alternative analysis of case study Arcen.

A.1. General information
A.1.1. Present levee structure
The area of Arcen is a typical example were levees are constructed after the floods of 1993 and 1995 (see paragraph 1.1). At this moment the levees are assigned as "primary flood defence systems" with a present safety standard of an annual probability of exceedance of 1/250. The total length of the system is 5 km existing of 7 subsystems and a total of 25 hydraulic constructions (e.g. outlet sluice, stop log system, pumping stations, etc.)

The levee consist mostly of a masonry wall with a number of demountable systems and multiple corners zigzagging along and through people's backyards (see figure A.2). The basic design of the masonry wall before the execution was a straight forward line along the river. Fast consideration and a small plan period resulted into a relative large acceptance of citizens preferences which primarily caused the large zigzagging effect. At this moment there is no data available, or persons who can state they are certain, in what conditions the foundation of the present walls are. There is a significant chance that somewhere sheet piles, which are present on drawings, are not on the required depth or present at all. Figure A.1 shows the available data of a cross section in the database of the Water board.

A.1.2. Present safety assessment
The ring-levee of Arcen has been analysed by Vilier (2013) during his Master thesis. The expected economical damage due to a flood wave with an annual probability of exceedance of 1/250 is approximately 71.8 million Euro. Furthermore the ring-levee is investigated by a Dutch project "Veiligheid Nederland in Kaart, VNK2" (flood safety inventory of the Netherlands) which analysed 58 ring-levees concerning their probabilities of a flood. The present safety assessment of Arcen has been evaluated and has approximately an annual flood probability of 1/20.

A flood in this ring-levee will cause an expected economical damage between the 40 to 140 million Euro. The expected number of people affected by the flood is estimated at 2300 where 0 to 15 people are actual fatal casualties (de Groot 2012, Kind 2011).

In the upcoming few years, plans will be formed to design and execute a new levee system providing a safety level with an annual probability of exceedance of 1/250. Some decades later the ring-levee
of Arcen should ensure the new safety philosophy. The governing annual flood probability within this research, simulating this new safety approach, is assumed to be 1/1250.

A.1.3. Governing levee-section(s) and cross-section(s)

Before and during the procedure of defining levee alignment options, the levee can be divided into sections each with their similar parameters (e.g. cross-section, present construction type, space, functions, etc.). Figure A.2 illustrate which sections will be investigated during this research. A high raised apartment building is present at the left hand-side of section 1 in where the flood defence is integrated. Increasing the flood defence entails a tailor-made solution in where aspects are involved which are hardly to find elsewhere (for instance the gates to regulate airflow inside the building do not fulfill the present safety standard).
A.2. Levee alignments

Between levee sections one and two a restaurant is present including an outdoor platform at the other side of the flood defence. During extreme high waters a stop-log system will be placed which disables the passage to this platform. Also here all possible solutions are tailor-made and depending a lot of the preferences of the owners. At the right hand-side of the section two a green levee is present.

Per section one governing cross-section is chosen. As there is a significant difference between the available space of sections one and two, each has its own cross-sections (There are examples in where two sections at different places have similar cross-sections). One has to consider the three-dimensional effect of the total area during the alternative analysis. There is always a possibility to separate a levee-section if there are to large differences to the cross-section chosen and the rest of the levee-section. Although the length a section haves should be taken as large as possible (or the number of sections as small as possible). At the end it make no sense to have many different types of levee designs over a limited length. Besides the increasing costs, is decreases the LNC-value and increases the difficulty to maintain properly. The governing cross-sections are shown in paragraph A.3.1 and A.3.2.

A.2. Levee alignments

Normally, one consider the whole ring-levee to identify levee alignment options. During this case study, only the levees are considered which are chosen beforehand. Options are included which decreases the discharge profile of the river while options at other parts of Arcen can be thought of which increases this profile (e.g. the top North part of the ring-levee). The options are displayed for all considered section of the ring-levee but should be analysed separately. Hence, section I should be analysed for options 0 to 3 as well as section II (inclusive the consideration the levees should be connected to each other in total. The four options are:

**Option 0: The same alignment**

It appears to be the case, that no full data is available to calculate whether the present structure is able to withstand the forces of the potential flood. Therefore, for now, it is assumed that the present situation should be strengthened and heightened.

**Option 1: Levee alignment at the river-side**

A possible option is to move the levee into the river-side such that a certain distance is created between the levee and houses (see figure A.3). One of the conditions were that a levee may not be created inside the discharge profile of the river as it could influence its hydrodynamic behaviour in a negative way. To compensate this loss, possibilities at the other side of the river are available, like decreasing the height of the agricultural area.

![Figure A.3: Levee alignment option 1; case study Arcen](image)

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option 2: Levee alignment at the other side of the houses (at the street)
Most houses along the river Meuse in Arcen are constructed many years ago (at least before 1993/1995). Some speak about the fact that some citizens are “used” to the situation a flood enter their homes and gardens. Keeping the visual effect of the river along their houses can be an important factor. Discussable is the idea to design a levee at the other sides of the houses like indicated in figure A.4. At this moment the houses are protected with a certain safety level (for instance 1/10 years), so the total damage which have to be included into the LCCA might result in a fair alternative.

At the other side of the houses, there is not much room either as can be seen in figure A.5. The inundation height of a flood will raise up to 0.5 a 1 meter from the street pavement. Due to the cultural value of the picturesque streets, alternatives can be thought of limiting the decrease of aesthetic value.

option 3: connect houses at the river-side
In option 3, houses will be connected at the river-side. Partly a watertight connection can be made with the existent masonry wall creating a wide enough flood defence which fulfil piping. Figure A.6 shows an example of the flood defence top view. A big disadvantage is the increase in corners, structures and doorways to close. Therefore it will not be a preferable solution for stakeholder such as the government. Nevertheless, it is technical possible and might be in accordance with residents preferences.

There are multiple ways to connect houses and create flood prevention. One has to consider all openings in a house including doors, windows, rainfall drainage, sewer system, ventilation points of a house, etc. When flood water stays for some time, the brickwork or stonework of houses has to be protected as well. There is a change flood water enters a cavity wall and creates leakage inside.

The drawn line in figure A.3 to A.6 indicate the basic alignment option. During the following steps levee design alternatives are formed wherein the exact alignment will be defined.
A.3. Available space

Next, all possible levee design options are gathered per levee-section and levee alignment option. Each levee design option is drawn (sketch design) inside the governing cross-sections based on the design criteria (see chapter 2). Whenever it turns out that there is not enough space available, the alternatives are assigned with "No". If the alternatives do not fulfil the design criteria and fits within the governing cross-section, it will lead to a "Yes". Here the assumption is made that it is not allowed to inter-phase the daily water level of the river Meuse.
A.3.1. Ring-levee section 1
This paragraph state all alternatives of levee section 1 together with a cross-sectional sketch design. The length/height ratio of all cross-sections is 1:2.

**Option 0: The same alignment; A: Structural design option**
A sheet pile will be placed and connected to the front of the existing levee structure. On top a concrete capping beam will be constructed. In a later stadium, one can include other structural elements such as a glass wall or a dynamic system. An illustration of the cross-section is presented in figure A.7.

![Figure A.7: Option 0: The same alignment; A: Structural solution](image)

**Option 0: The same alignment; B: Parallel green levee**
An illustration of the cross-section is presented in figure A.8. It is clearly visible that this alternative does not fit properly. Therefore a "NO GO" is given.

![Figure A.8: Option 0: The same alignment; B: Parallel green levee](image)

**Option 0: The same alignment; C: Double wall system**
A sheet pile will be placed at the river-side of the present situation. On top of the present situation, the masonry wall will be increased with one meter. A width of 4,5 meter is used to create a maintenance- and inspection path. The sheet pile top will approximately reach 5 meters above surface level. An illustration of the cross-section is presented in figure A.9.

**Option 0: The same alignment; D: Double wall system with a structural design option**
A sheet pile will be placed at the river-side of the present situation up to 0,80 meters above ground level from a house perspective. soil will be used to fill up the space between the sheet pile and existing

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A width of 4.5 meters is assumed to create a maintenance- and inspection path. The sheet pile top will approximately reach 4 meters above surface level. On top a concrete capping beam, or retaining wall will be constructed. In a later stadium, one can include other structural elements such as a glass wall or a dynamic system. An illustration of the cross-section is presented in figure A.10.

---

**Option 0: The same alignment; E: A green levee**

An illustration of the cross-section is presented in figure A.11. It is clearly visible that this alternative does not fit properly. Therefore a "NO GO" is given.

**Option 0: The same alignment; F: A green levee with a single structural design option**

The sheet pile should be placed somewhere at the riverside. The smallest height of the sheet pile above ground level is approximately 3 meters at the interface where the daily waterline meet the embankment.

**Option 1: Levee alignment at the river-side; G: Green levee**

This alternative has the same (or almost the same) cross-section as alternative B in this ring-levee section. Therefore a "NO GO" is assigned to this alternative. During this stage of the screening process, no further thought are made regarding the three-dimensional effect. It might be the case that this alternative is applicable to most of the section, but not to this part (where cross-section two is drawn). However, this alternative always entails the fact that there have to be a big change of alternatives within this section. Hence, having large alternative changes in one section is unwanted or one has to redraw the sections. Therefore, this alternative is assigned as a "NO GO".

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Option 1: Levee alignment at the river-side; H: Green levee with a single structural design option
The same consideration can be made as alternative G as this one can be assumed to be almost equal to alternative E. Therefore a “NO GO” is assigned to this alternative.

Option 1: Levee alignment at the river-side; I: Double wall system
The same consideration can be made (see alternative G) as this one looks a lot like alternative C. Therefore a “NO GO” is assigned to this alternative.

Option 1: Levee alignment at the river-side; J: Green levee with a structural design option
An illustration of the cross-section is presented in figure A.13. It is visible that this alternative does not fit properly. Therefore a “NO GO” is given.

Option 1: Levee alignment at the river-side; K: Structural design option
This alternative does fit within the “boundaries” (see figure A.29) but has the same basic idea as alternative D. However, there are other cross sections within this levee section in where there is more room available. Hence, this alternative should get a GO for now.

Option 2: Other side of the houses; L: double wall system (Raising the street)
the street width is estimated between the 7 to 13 meter. Within this alternative, the street has to be changes in a one direction only (which is a huge plan as line busses are using this road also). Secondly, the sewer system and other cables have to be removed/renewed at some locations. The sewage near
the houses along the river need extra sewage defend valves. Additionally one has to redesign the drainage system of the rainfall on top of the street. At last the cultural picturesque view of the street will be damaged.

**Option 2: Other side of the houses; M: Dynamic system raising out of the street**

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A dynamic system limit the visual impact in contrast to alternative 2; L. Also here, carefully consideration have to be taken how to deal with the sewer and cable system underneath the road and walking paths. Secondly, the system have to be installed, creating a pipe to use the flood water or connect the system electronically which enhance extra project costs as well as maintenance.

**Option 2: Other side of the houses; N: Connecting houses including a structural design option against seepage**

By connecting houses in the longitudinal direction along the river, and making the houses flood preventive, a flood defence can be created. A sheet pile, or a similar system, can be placed in front of the houses prevent undesirable seepage. The system will be connected to the foundation of the houses creating an more ore less impermeable flood defence. It is doubtful if this alternative is interested considering the construction way. Executing a sheet pile in front of (old) houses should always be considered with great attention. Secondly, the failure of closure can be relative large due to many different openings which have to be closed.

**Option 3: Connect houses at the river-side; O: Connect houses, use structural design option against seepage**

BY connecting houses in the longitudinal direction along the river, and making the houses flood preventive, a flood defence is created. A sheet pile which will be placed in front of the houses prevent undesirable seepage. The sheet pile will be connected to the foundation of the houses. It is doubtful if this alternative is interested considering the construction way. Executing a sheet pile in front of (old) houses should always be considered with great attention. Secondly, the failure of closure can be
relative large due to many different openings which have to be closed. With the latter aspects keeping
in mind, the project costs are estimated. When the resulting costs are relatively high, this alternative
will not be looked at further.

Figure A.18: Option 3: Connect houses at the river-side; O: Connect houses, use structural design option against seepage

Option 3: Connect houses at the river-side; P: Connect houses and create a watertight construction to the present flood defence
Instead of using sheet piles or a similar system, like alternative (3;O), the present situation can be used. In this case, the floor between the houses and the present levee structure needs to be constructed watertight. At his stage, it is assumed to use concrete.

A.3.2. Ring-levee section 2
This paragraph state all alternatives of levee section 2 together with a cross-sectional sketch design. The length/height ratio of all cross-sections is 1:2.

Option 0: The same alignment; A: Structural design option
A sheet pile or similar structural system will be placed in front of the existing structure and connected to it. On top a concrete capping beam will be constructed. A structural element, consisting of concrete, glass, masonry, steel or a combination can be placed on top of it. An illustration of the cross-section is presented in figure A.19.

Figure A.19: Option 0: The same alignment; A: Structural solution

Option 0: The same alignment; B: Parallel green levee
An illustration of the cross-section is presented in figure A.20.
Option 0: The same alignment; C: Double wall system

The present levee will be heighten (or a new wall will be placed in front of it) and a second wall will be placed at the river side. Earthy material will be placed between the walls. The minimal width of 4,5 meter is used to establish a maintenance- and inspection-path. The height between the wall's top and surface is approximately 2,5 meters at the river-side and 1,8 meter at the land-side. A sketch illustrate the cross-section of alternative C in figure A.21.

Option 0: The same alignment; D: Double wall system with a structural design option

This alternative decreases the height of the double wall system by placing a structural design option on top (see figure A.22). It provide options to have a not to high structural system inside some one’s backyard. Subsequently it might reduce major maintenance costs.
Option 0: The same alignment; E: A green levee with a structural design option
The same advantage holds to include a structural design option on top of a green solution as alternative D. Figure A.23 illustrate a sketch of the cross-section.

![Figure A.23](image)

Option 0: The same alignment; F: A green levee with a single structural design option to reduce width
In contrast to the latter alternative, this one has a structural solution to reduce the width and to decrease possible seepage of a parallel levee alternative. Although this alternative give perspective in levee-section 1, here it loses its privileges as more space is available. Secondly it creates potentially dangerous situations when having a slope at the side of a vertical wall. However, it still fits the available space and is an alternative. A cross-sectional sketch design is illustrated in figure A.24.

![Figure A.24](image)

Option 1: Levee alignment at the river-side; G: Green levee
A green levee at the river-side of the present situation does enhance advantages for the backyard owners. Although it requires a large area which has currently a function of a flood-plane. Figure A.25 illustrate a cross-section of alternative G.

Option 1: Levee alignment at the river-side; H: Green levee and single structural design option
A green levee with a single wall at the river-side of the present situation with a gentle slope in the backyard. This levee design alternative reduces the hydraulic impact with respect of alternative G. Figure A.26 shows the cross-section of this alternative.

Option 1: Levee alignment at the river-side; I: Double wall system
The double wall system will be created further away of the houses (see figure A.27). Although the visible impact decreases from the owners viewpoint, the privacy increases as people who are recreating are not looking directly inside the backyard. The inspection path will be constructed on top of the flood...
Figure A.25: Option 1: Levee alignment at the river-side; G: Green levee

Figure A.26: Option 1: Levee alignment at the river-side; H: Green levee and single structural design option

defence such that it can be used during all circumstances.

Figure A.27: Option 1: Levee alignment at the river-side; I: Double wall system

Option 1: Levee alignment at the river-side; J: Green levee with on top a structural design option
Alternative J has the principals of alternatives E and G combined (figure A.28).

Option 1: Levee alignment at the river-side; K: Structural design option
A structural design option at the river-side at first hand doesn’t seems to be the first thought. Regarding the visual impact, and the decrease in river flow profile, this alternative is not preferred with respect to alternative A. However, it provide a long horizontal backyard which entails some advantages. This can be seen in figure A.29 where a sketch design of the cross-section is drawn.

Option 2: Other side of the houses; L: Raising the street

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There is certainly no space here to raise the street. One of the design criteria, is that a wheelchair should be able to enter the houses on both sides of the street. This alternative get a NO GO for levee section 2.

**Option 2: Other side of the houses; M: Dynamic system raising out of the street**

Figure A.30 illustrate the dynamic system in the governing cross-section. Sewer and cable systems underneath the road should be considered with great care when continuing the investigation to this alternative. All connections with houses to the sewer c.q. other underground connections, are all possible threats. Leakage has to be prevented via help of a second sewer system and/or one-direction valves. Moreover, the system has to be installed electronically or with inlets which increases difficulty of maintenance and inspection.

**Option 2: Other side of the houses; N: Connecting houses including a dynamic closing system**

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By connecting houses in the longitudinal direction along the river, and making the houses flood preventive, a flood defence can be created. A sheet pile, or a similar system, can be placed in front of the houses prevent undesirable seepage. The system will be connected to the foundation of the houses creating an more or less impermeable flood defence. It is doubtful if this alternative is interested considering the construction way. Executing a sheet pile in front of (old) houses should always be considered with great attention. Secondly, the failure of closure can be relative large due to many different openings which have to be closed.

**Figure A.31: Option 2: other side of the houses; N: Connecting houses including a dynamic closing system**

**Option 3: Connect houses at the river-side; O: Connect houses, use structural design option against seepage**

Houses will be connected in longitudinal direction with several doorways within the flood defence. By creating a more or less impermeable connection with the present situation, one is able to limit seepage. Figure A.32 shows the sketch design inside the governing cross-section.

**Figure A.32: Option 3: Connect houses at the river-side; O: Connect houses, use structural design option against seepage**

**A.4. Cables and pipes**

With help of GIS software and available data, an indication of the present cables and pipes are shown in figure A.33. Especially many cables/pipes are present underneath or along the road in section 1. Although, there are probably more cables and pipes present, it gives some understanding about what priority types and number of pipes and cables are present. Unfortunately no sewer-system or other types of cables and pipes are (except gas) indicated in the top part of the figure while the houses are provided with energy, water and sewer. Therefore it is assumed that at least a sewer system is present underneath the road on the East side of the houses along the river Meuse.
Figure A.33: Indication of the present cables and pipes in Arcen (source: WPM)
Levee design types examples

The purpose is to define a wide range of levee design options, which might be used in a flood defence project. This appendix give an overview of present-day levee design solutions which can be used in the process of defining and screening them during a case study. Solutions for flood hazards are categorized to identify the area the alternatives can be used. for instance, agriculture areas are often defended by green alternatives\(^1\), while urban areas often struggle with limited space leading to a more structural solution.

There has been a significant development of flood defence systems over the past few years (Ogunyoye \textit{et al.} 2011). Usage of building (e.g. houses with a flood defence function like Kampen, the Netherlands; WGS 2007) as well as innovative temporary, demountable or self closing dynamic systems. Many information is retrieved from Ogunyoye \textit{et al.} (2011), van Heereveld (2008), Blaauw \textit{et al.} (2012) and websites of various suppliers. An overview is given in table

The first paragraph state the green design option where one could think of during an alternative analysis. Paragraph B.1 describes the permanent structural design options while paragraph B.2 elaborate the temporary and demountable design options.

\(^1\)The Netherlands describes a green alternative as an earthy levee with mainly a grass cover which is one of the most applied solutions in the country.
Levee design types examples

Table B.1: Alternatives overview of the types, design option and examples

<table>
<thead>
<tr>
<th>Type</th>
<th>Design option</th>
<th>Examples</th>
</tr>
</thead>
<tbody>
<tr>
<td>Green</td>
<td>Square rising</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Increase one side</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Parallel levee</td>
<td></td>
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**B.1. Green design options**

In landscape areas, a green design option is an obvious possibility as it is relatively cheap and easy to inspect and maintain. One speaks of a green levee when it consists of only earthy material often with a top-layer of clay and a cover of grass. Sometimes the full core of the levee consists of clay. Many different shapes are present depending on its history, wave and wind action, infrastructure and spatial planning. The green design option requires significant more room with respect to a structural one.

To help the process of defining and screening alternatives, some standard levee options, including some illustrations, are presented below. In these cases only a reinforcement or upgrade of the levee is presented. In the case of some levees along the river Meuse in Limburg, total new levees have to be constructed. It is obvious that the same profile can be chosen as solution. Furthermore a few solutions can be combined in order to strengthen and/or heighten the levee (e.g. making a gentler slope or creating a berm).

**Square rising**

A common solution of reinforcing the levees is raising it vertically (sometimes referred to as square rising), which is illustrated in figure B.1. The levee will be raised symmetric. If the crest-to-crest width...
is too small, both slopes will be increased (sometimes the slope(s) are flatted to increase strength or more steeper to keep the same width at the toes of the levee).

**Increasing one side**

Next, the levees could be raised more land-inward (figure B.2) due to the restriction no upgrade is allowed at the river-side. Unfortunately, it requires more space at the land-side but still is an often applied upgrade. Of course the one could raise to the river-side as well, which is the same type of solution.

**Parallel levee**

A recognized solution is a parallel levee which changes the crest position to one side and creates a “new”, higher levee on the inner or outer slope of the old one (ENW 2007). The advantage of this measure is that existing objects like roads or trees, can stay on top of the levee. Secondly, in many cases in Limburg, the ground level at the land side is very high. So creating a levee such as illustrated in figure B.3, might be cheaper in contrast to a measure at the river-side.

**Berm**

Sometimes a levee only need to be strengthen to create more stability or to increase resistance against seepage. A berm is an well known solution to take away the problem. At this moment, it is an often applied solution due to new insight in the piping and heave phenomenon. The berm, illustrated in
figure B.4 is situated on the land-side, but again, can be constructed on the river-side.

**gentler slopes**

Figure B.5 illustrate the a solution to create a more gentler slope. In this case, the slope is made more gentler by taking away material on the top part of the slope which can be used on the bottom part such that the total river flow profile is guaranteed. A more gentler slope result in a more stable slope and increase resistance against seepage. Secondly, it is easier to maintain the grass cover by mowing with a machine ore the use of herbivores such as a sheep.

**continuing living layer**

In order to keep the levee system as a thin structure to preserve landscape appearance, one could extend the berm solution with very gentler slopes (e.g. 1:20) which can be as an other function, for instance as farmland. The total levee looks relatively small while even more width is present in contrast to the solution of a simple berm. An illustration is presented in figure B.6. This solution might be a good idea when flood defence managers discover possible hindrance with land owners. One has to keep in mind how to cope with the rainfall as well as the moisture content of the ground. Secondly, criteria have to be assigned with the land owner concerning the maximum size of a crop/tree to plant and the way it has to be maintained.

Deltadike

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Another earthy type of levee, which is under discussion is a "Deltadike"\(^2\). This robust design has a certain height and strength which almost neglects the probability of a flood. During the case studies the Deltadike is not an logical solutions and probably not in favour of any stakeholder due to the high costs and the big changes into the environment. When government or project investors consider new districts along a river, this solution is more appropriate. Example of a constructed deltadike are Japan or the Wantij in Dordrecht, the Netherlands (Silva and van Velzen 2008).

\(^2\)Sometimes referred to as "climate dike" (Dutch deltadijk en klimaat dijk)

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B.2. Structural design options

Structural design options are often considered when one has to deal with urban areas or limited space. Sometimes it is chosen to apply (demountable) structures if they only have to be applied for a few years (like the stop-log system in areas of water-board Peel and Maasvallei). The structural design options are divided into three categories: Permanent-, demountable- and temporary systems in which the permanent options are most acknowledged and proven techniques. Different researches has been carried out to state types of temporary and demountable systems such as the "Temporary and Demountable Flood Guide" (Ogunyoye et al. 2011) or the "Choice-model for temporary and demountable levee systems" (van Heereveld 2008). Both shown interesting design options as flood defence. Sometimes it is not clear when a structure belongs to the demountable- or the temporary category. During this research a temporary structure will only be used for emergency purpose or as secondary structure.

B.2.1. Permanent design options

In this setting a permanent structural design options is a hard structure without any demountable or temporary elements within its flood defence system. There are many alternatives where can be thought of with green and a structural solutions thinking about L-shaped concrete walls, sheet piles with a concrete capping beam or concrete blocks. Due to the latter fact, it is difficult to achieve standardized solutions. Hence, examples shown below are very neutral. The International Levee Handbook, or ILH (CIRIA et al. 2013) describes various alternatives with structural types together with a green solution. This report is used to form the solutions described below.

In many cases, combinations of these solutions can be thought of. For instance, a glass wall element on top of an L-shaped concrete block with a sheet pile preventing redundant seepage.

Retaining wall

A wall on crest of levee is an obvious solution. There is a wide range of (prefabricated) retaining wall element types available in the today’s marked (Morris et al. 2007). Figure B.8 illustrate an L-shaped concrete retaining wall element on top of an excising green levee. All figures are illustrated with a green present situation which is not always the case with structural solutions. Advisable is to sketch each possible solutions in a cross-section of the present situation to see if the solution is suited.

![Figure B.8: Permanent design option; retaining walls](image)

Figure B.9 illustrate examples of other elements which are introduced by the ILH (CIRIA et al. 2013). All types need an foundation which sometimes includes an wall within the earthy structure to prevent seepage. The type and material also depends on the present situation. A masonry solution can be preferred nearby a middle-age boulevards with an old masonry citywall.

![Figure B.9: Concrete retaining elements. From left to right: L-shaped wall; Concrete or built wall; inverted T-wall; Massive built, concrete or gabion box wall.](image)

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3Dutch: Keuzemodel tijdelijke en demontable waterkeringen

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**Double wall system**
Heightening the levee with limited space and/or with a road on the levee crest, can be done with a double wall system. A sheet pile or concrete element will be placed at the crest with earthy material between it. Anchors can be used as possible measures to increase stability.

![Figure B.10: Permanent design option; levee crest raising with mechanically stabilize earth](image)

**Single sheet pile**
A single sheet pile, shown in figure B.11, can be used when a green solution is preferred but not allowed to increase further to the riverside. The position of the sheet pile (or other types like a diaphragm wall) could differ depending on the situation. When for instance, the levee only need to be strengthen, a sheet at the berm could help. When considering a green alternative, using a sheet pile is much more expensive in contrast to executing a gentler slope or an extra berm. The illustrates solution does not differ much with regard to the retaining wall one.

![Figure B.11: Permanent design option; levee crest raised with sheet piling](image)

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*Sometimes referred to as coffer dam (Dutch: Kistdam)*

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Glass wall
A more innovative solution is the use of glass as flood defence (figure B.12). In contrast to alternatives with structural elements such as concrete and steel give a glass wall still some connection to a river or lake. During the design process, some insight have to be given if it can withstand all acting forces such as the water pressure, ice loads and collision of storm debris. The glass panels, sometimes consisting of multiple layers of glass, are mounted together by steel or concrete pillars which transport the loads to the foundation. Vandalism leading to a crack or graffiti are also aspects which have to be analysed.

A glass wall has been used in Breskens (the Netherlands) to reduce overtopping and overflow. Another glass wall is constructed in Well, United Kingdom (see figure B.13) and 36 meter has been constructed in Roermond (The Netherlands) at the boulevard. The latter flood defence interface the reference high water level of the river Meuse.

Advantages:
- Permanent flood defence system;
- minimal visual impact;
- can be mounted onto suitable existing walls or foundations;
- relatively minor maintenance;
- no storage space necessary;
- no size restriction (can be used up to 3000 mm).

Disadvantages:
- glass have to be cleaned;
- possible to reflect sunlight which decrease safety (vessels, cars);
- vulnerable for collision impact;
- protection height is hard to increase.
Dynamic system

Another innovative flood defence group are the dynamic systems. During normal circumstances they are closed and mostly underground. When an extreme high water occurs, they will be lifted via pneumatic or hydraulic lifting tool by hand, electronic or automatic. Figure B.14 illustrate the basic idea of the permanent dynamic systems. Some systems are lifted via the upward force of the water. Others are hinged and rotate from a horizontal to a vertical position. Over the literature, some design options are categorized as demountable, fully pre-installed systems (Ogunyoye et al. 2011).

Examples of a dynamic, self-deploying systems are the Self-Closing Flood Barrier, the Vlotterkering, the floodbreak, the Extra and the Flip-Up flood barrier.

Self-closing Flood Barrier

This vertically floating gate is a fully pre-installed flood defence system installed in the ground. When the water rises to approximately 10 centimetre below the flood level, a basin starts to fill. The gate will rise vertically and will be locked into position making it watertight. The flood water can now continue to rise without flooding the protected area (van den Noort 2009). Figure B.15 shows the use and installation of the Self-closing Flood Barrier. The cover picture illustrate the installation in Workington(UK) as well.

Facts:

- Approximate time of deployment: rises with water level;
- approximate costs, 1 meter high: € 3.820 m';
- maximum standard protection height to 2,5 meter

Advantages:

- Fully pre-installed solution;
- minimal impact view;
- no storage area necessary;
- system height up to several metres;
- do not depends on flood warning system;
- limited in width.

Disadvantages:

- Protection height hard to increase;
- possibly sensitive for debris who can jam the system;
- possibly sensitive for settlements;
- possible to stuck during cold weather conditions;
- when system failure occur, not much time left to create an alternative flood defence.

This innovative variant hasn't been applied in the Netherlands yet. The main critical subject regarding these kind of dynamic systems is the failure to close. A reliability analysis has been carried out for this specific system, the self closing flood barrier (van der Linde and Voortman 2012). The analysis, in accordance to the Dutch present statutory assessment, graphically shows under what probability of
failure and closure frequency the defence satisfies. Figure B.16 illustrate when the barrier complies with the adjustment of a second intake pipe between the service pit and the river. Location specific aspects influence the results and are not considered during this analysis. Moreover, the level of analysis detail is limited. Although this limitation, it does give an view to the likelihood the barrier will fulfil a more detailed analysis.

Also reliability of the system in accordance of the new safety approach is checked roughly. An assumption has been made that the maximum allowable probability of failure given a certain probability of exceedance has to be 10 times smaller (e.g. annual probability of exceedance of 1/250 result in a maximum closure frequency of 5. The maximum closure frequency will be 0.5 times per year average.
when applying the new safety approach).

**Vlotterkering**

The Vlotterkering is a Dutch new flood defence system, invented by Mr. Jansen and Mr. Vermond as a solution to store water. It was the winning design for a price-question written out by the water board Delfland. A concrete bin will be installed with, inside a steel container hinged at the inner side of the concrete bin. During high water, the bin starts to fill with water. Due to Archimedes forces the steel container raises. when the water level increase above the bin, the container in combination with the hinge and skirt (riverside) retain the water. Figure B.17 illustrate the vlotterkering at Venlo along the river Meuse in Limburg closed. Figure B.18 illustrate the same situation open.

Currently the system has been tested and engineering agencies and institutes are investigating the probability of failure with this system also in combination with the new safety philosophy.

**Facts:**

- Approximate time of deployment: rises with water level;
- approximate costs, 1 meter high: € 6.000 - 8.000 m’;
- maximum standard protection height to 2 meter (can be enlarged)

**Advantages:**

- Fully pre-installed solution;
- minimal impact view;
- no storage area necessary;
- system height up to several metres;
- do not depends on flood warning system;
- multiple ways to open (pump system, inflow through an inlet and overtopping)

**Disadvantages:**

- Protection height hard to increase;
- possibly sensitive for settlements;
- when system failure occur, not much time left to create an alternative flood defence.

**Floodbreak**

The floating gates system is installed into the ground and ready to deploy without any human interventions. The gate starts to floats when water reach a certain height until it stands perpendicular to the ground. Figure B.19 shows a picture of the flood-break gate system.

**Facts:**

- Maximum standard protection height to 3,6 meter.

**Advantages:**

- Fully pre-installed solution;
- minimal impact view;
- no storage area necessary;
- system height up to several metres.

**Disadvantages:**

- Protection height hard to increase;
- sensitive for debris who can jam the system;
- possible to stuck during cold weather conditions;
- needs hard structure sides;
- sensitive for overtopping;
- when system failure occur, no time left to create an alternative flood defence.

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Flip-Up flood barrier

In contrast to the other variant, this system can be raised by hydraulic pumps which can be activated by a push button, automatically triggered by sensors or manually. The barrier lays horizontal under normal conditions and stands vertical when activated. The hinges are connected at the land-side hand under

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the gate elements. Hence, during high water the hydraulic/electric system is exposed to the flood water.

Facts:
- Maximum standard protection height to 2 meter.

Advantages:
- Fully pre-installed solution;
- minimal impact view;
- no storage area necessary;
- system height up to several metres;
- can be manually operated during power failure;
- can resist traffic loads on top of the inactivated system;
- can be activated whenever wanted.

Disadvantages:
- Protection height hard to increase;
- possible to stuck during cold weather conditions;
- many maintenance after high water;
- connection between gates is an issue;

Extra (Self Extracting water barrier)
The Extra is a fully pre-installed flood defence system which raises automatically when (flood) water enters the bin. It is the same type of system as the SCFB but has, if preferred multiple walls opening on different water levels.

The Extra has been tested two days by the research institute Deltares. The facility tests with configuration of multiple water heights and different wave attacks. According to the promoting company the results showed excellent operation, rigid and stable (Grontmij nd).

Facts:
- Approximate time of deployment: rises with water level;

Advantages:
- Fully pre-installed solution;
B. Levee design types examples

Figure B.20: Flip-Up flood barrier system, retrieved from floodcontrolinternational.com

- minimal impact view;
- no storage area necessary;
- system height up to several metres;
- do not depend on flood warning system;
- multiple ways to open (pump system or flow through a inlet)

Disadvantages:

- Protection height hard to increase;
- possibly sensitive for settlements;
- when system failure occur, not much time left to create an alternative flood defence;
- not tested on actual scale (as far one knows of), yet.

Figure B.21: Impression Extra flood defence system

Block-it

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The block-it system is a concrete wall located underneath the ground level which can be raised with help of hydraulic jackets. The system originally is developed to prevent ram raids. The system can be made fully automatic and operated elsewhere if preferred. At the moment, the company is continuing their investigation to the probability of failure regarding flood events.

Facts:
- Approximate time of deployment of a few minutes;
- Expected costs around the range of € 6,000 - 8,000 (Source: Maarten Eickholt);

Advantages:
- Fully pre-installed solution;
- minimal impact view;
- no storage area necessary;
- system height up to several metres;
- relatively easy to test.

Disadvantages:
- Protection height hard to increase;
- possibly sensitive for settlements;
- when system failure occur, not much time left to create an alternative flood defence;
- hard to inspect with the naked eye.

Figure B.22: Illustration of the block-it system working as a coupure (Retrieved from: blockitnow.nl).

Geotextile preventing piping
There are various techniques to reduce the probability of piping and/or heave. A general, often applied variant a berm at the inner slope (described in paragraph B.1). A new innovative idea is the use of a vertical open geotextile at the tone of the inner slope, creating a filter reducing the formation of a pipe. At this moment a water board in the Netherlands investigate different aspect like reliability, constructibility and costs in cooperation with companies and institutes. Figure B.23 illustrate the vertical open geotextile in a dike cross-section.

Seepage occur at a levee structure creating a water flow through the earthy material. When the current is stronger than the friction of a sand particle, the particle starts to move. The total friction of the current drops resulting in more particles moving to th surface. Hence, a pipe which could end up to the river side and undermine the structure. In the case a vertical geotextile is places, the particles flow stops at that position. The seepage of course continues.

text about present technology...

Facts:
- only a measure to reduce piping;

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• maximum depth of 6 to 8 meters;
• estimated costs for 3 meter depth: € 50,-.

Advantages:
• Sometimes easy to execute;
• execution most of the time relatively fast;
• low costs;
• no hard elements in the ground;
• limited excavation.

Disadvantages:
• Can only be used whenever a road or path is present;
• hard to used nearby cables and pipes;
• still some seepage;
• not a full proven technology yet.

**B.2.2. Demountable design options**

Demountable systems consist of pre-installed elements at the location of the flood defence, and elements stocked elsewhere which can me mounted on the pre-installed parts. Those elements have to be transported to the flood defence during extreme high water event. There are lots of demountable systems available nowadays each with its pro’s and con’s (Ogunyoye et al. 2011, van Heereveld 2008, Peijnenborgh et al. 2006). Some demountable systems can be fully installed at the location of the flood defence itself and are not very clear to be a permanent or demountable system. The deviation partly depend on the closure time for the system weather to be a demountable- or a dynamic permanent pre-installed system.

**Free-standing barriers**

Figure B.25 illustrate the basic idea of a free standing barrier. The top and bottom illustration of the figure shows the Free-standing barrier during normal circumstances while the other one shows the close barrier during extreme high water conditions. These barriers are made of sections which are joined together to form a continuous watertight barrier. The ground fixing connection and foundation
Figure B.24: Permanent solution; execution of closed geotextile (retrieved from: lareco.nl)

are the only permanent parts of this solution. During an extreme high water, a pane can be placed and connected to the foundation. Next, one can unfold the barrier to an L-shaped structure. The weight of the water should provide a water tight seal between the demountable part and the ground surface. Some free-standing barriers are rigid and can’t be unfold, but does require more stocking space. (Ogunyoye et al. 2011)

Figure B.25: Demountable solution; free-standing barrier

Aquafence

The aquafence system is a foldable, temporary structure consisting of two panels. Installed, the panels are L-shaped (connected wit an hinge). The structure is stable due to the water pressure on the horizontal panel. During installing, besides the closing of the panels and the shores, one is able to close the connections. Figure B.26 shows the aquafence somewhere in Norway and the installation of the system in New York(USA).

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Facts:
- Approximate time of deployment: 1 hours for 100 meter with 6 to 8 persons;
- approximate costs of a 1 meter high barrier: €611/m';
- maximum standard protection height of 1,8 meter.

Advantages:
- Relatively low visual impact;
- relative low equipment requirements for installation;
- easy to clean and reuse.

Disadvantages:
- Protection height is hard to increase;
- storage area necessary;
- long installation and mobilisation period;
- expensive to test;
- significant seepage may occur under the barrier in uneven terrain;
- failure has occurred, which indicate a relatively high risk (Halle, Germany).

Aqua-barrier (fixed)
The rigid Aqua-barrier is a hollow polyethylene demountable flood barrier which can be mounted to its foundation with bolts. The barrier will fill itself with flood water through holes at the front face. A total loaded weight of 3,2 tons prevent the system of sliding. When empty, one element only weights 85 kg. rubber seals are creating a water-thigh connection between the elements itself as well as to the continues flood defence at their sides. A pre-installed foundation works is required to install the system. The installation of the fixed Aqua-barrier and the usage are shown in figure B.27.

Facts:
- Approximate time of deployment: 5 hours for 100 meter;
- approximate costs of a 1 meter high barrier: €12.000/m';
- maximum standard protection height to 1,5 meter.

Advantages:
- Relatively low visual impact;
- durable;
- easy to install on existing foundation;
- easily cleaned and reused.
Disadvantages:

- Very logistic / heavy transportation requirement;
- Large storage area necessary;
- Long installation and mobilisation period;
- Available systems have fixed heights;
- Very costly;
- Need multiple persons for installation;
- Sensitive for settlements;
- Expensive to test.

**Frame barrier**

The partly pre-installed frame barrier, or stop-log system is a widely used solution. The barrier consists of rigid panels which can be placed horizontally between mounted vertical pillars. The pillars are connected to the foundation. Seals at the panels and in some cases the pillars prevent possible seepage. This type of demountable solution can be used for temporarily increasing the flood defence (e.g., when the maximum water level of a flood wave is known, one can choose to close only the first half panels of the barrier if it fulfills the forecasted maximum). Figure B.28 illustrates this type of flood defence barrier. The top figure shows a cross-section during normal circumstances, the flood defence is not installed yet. The lower figure illustrates the installed closed Frame barrier during high water.

At this moment more than 4 kilometer of this system is applied as primary flood defence system along the Meuse in Limburg. The probability of failure due to the fact that it can’t be closed (on time) is an ongoing issue and discussion as stated before. Some difficulty can occur with other systems described in this section. Nevertheless, when the total amount of this system is limited, it can be a good solution. Secondly, new insights into this system or the way to analyse and calculate the probability of failure may end up with a positive result.

**Stop-log system**

The stop-log system is a demountable, pre-installed, flood defence consisting of aluminium (or other type of steel) beams and centre support posts. These posts are mounted on a concrete foundation every two or three meters. In case of a potential flood, the beams are stacked between the centre support posts to create a flood defence. Figure B.29 shows an illustration of the stop-log system. Figure B.30 shows the installation of a stop-log system in the United Kingdom. It is a widely used (Venlo (NL), Kampen (NL), Bewdley (UK), Dresden (GE), etc.)

**Facts:**

- Approximate time of deployment: 2 hours for 100 meter;
- Approximate costs of a 1 meter high barrier: € 160 - € 275/m;
- Maximum standard protection height to 5,0 meter.

Figure B.27: The usage of the Aqua-barrier is shown on the left hand side, the installation on the right.
Advantages:
- Relatively low visual impact;
- protection height can be increased;
- durable;
- easy to install on existing foundation;
- system height up to several metres.

Disadvantages:
- Very logistic / heavy transportation requirement;
- large storage area necessary;
- long installation and mobilisation period;
- sensitive for settlements;
- expensive to test.
B.2. Structural design options

Section barrier
The section barrier has sections made of rigid material which can be interlocked in each other to form a continuous flood defence. The barrier is fully pre-installed which is an advantage in contrast to the frame barrier. Under normal circumstances the barrier is installed inside the ground. Figure B.31 shows the idea of the section barrier close and open. Some types include pillars to strengthen the barrier and resists the acting forces of the water pressure and possible storm debris.

Dutchdam
These type of system consist of multiple sections (most of the time steel or fibreglass elements), which can be connected to form an continuous flood defence. All sections are integrated in the system. This system is applied in Ireland, Australia, United Kingdom, The Netherlands and France (Silvester 2011). Figure B.32 shows an illustration of the stop-log system.

Facts:
- Approximate time of deployment: 2 - 4 hours for 100 meter;
- approximate costs of a 1 meter high barrier: €750/m';
- maximum standard protection height to 2,5 meter.

Advantages:
- Minimal impact on view;
- no external storage requirement;
- no transportation is required;
- system height up to several metres.

Disadvantages:
- Protection height hard to increase;
- sensitive for debris who can jam the system;
- possible to stuck during cold weather conditions;
- sensitive for vandalism;
- sensitive for robbery.

B.2.3. Temporary design options
A temporary flood protection system consist only of removable products that can be fully installed during an flood event and removed completely afterwards. Most temporary system does not need permanent changes inside the quay or embankment for installation. It is stocked elsewhere and installed at a certain time depending on the water level forecasting. Temporary flood defence systems are most of the time not desirable as long term solutions as they entail a relatively large probability of failure.

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When an alternative doesn’t fulfil the required probability of failure, once can consider to use a second

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independent system such as these. For instance, when having a demountable system which is not reliable enough, the probability of failure of the system can be decreased by placing a temporary solution behind it. One should consider in what frequency this measure might be wanted. For example, installing both a demountable and a temporary flood defence system is very expensive. Hence, constructing this type of alternative at a height the have to be installed with a probability of occurrence of once every year might not be ideal. When the only have to be installed once every 10 years, the alternative starts to be more interesting. Besides accounting the frequency of installation the flood defence has to be tested as well. In the Netherlands, a test frequency of 5 years is demanded in order to check and practise the barrier system as well as the organisation.

**Tubes**

This temporary flood defence system is typically made impermeable geo-membrane or reinforce PVC materials. The can be rolled out as a long tube and filled with water/air to form a dam. Figure B.33 shows a cross-section of the basic idea. With some types multiple tubes can be installed forming a sort of pyramid. They are usually anchored down with pins or the use of an extended skirt at the river-side. Some types are provided with a membrane in the tube which prevents rolling over. A bid disadvantage is the vulnerability to sharp objects, one small tear or puncture result in a possible failure.

![Temporary solution; air or water filled tube](image)

**AquabARRIER (flexible)**

The AquabARRIER is a tube consisting of a nylon reinforced material. It can be rolled out during a potential flood hazard and filled with nearby water. The inner baffle inside the tube prevent the barrier from rolling over landwards. The AquabARRIER is illustrated in figure B.34.

**Facts:**

- Approximate time of deployment: 1 hours for 100 meter;
- approximate costs of a 1 meter high barrier: €50/m';
- maximum standard protection height to 1,80 meter.

**Advantages:**

- Quick and easy to install;
- relatively small storage;
- not location specific (each location do need a certain available width);
- possible to repair during service;
- easy to reuse.

**Disadvantages:**

- No long term solution;
- vulnerable for leakage;
- not suited well for large retaining heights;
• protection height is hard to increase;
• require flat surfaces;
• small sharp object could lead to major system failure;
• large site space required.

Figure B.34: Illustration of a tube filled with water on the left hand side, on the right the flexible Aqua barrier (van Heereveld 2008)

Tigerdam
The tiger dam is the same water-inflatable system as the flexible Aqua-barrier. It consist of multiple tubes which can be joined together relative quickly. When the tubes are stacked to increase the retaining height, the barrier has a pyramid shaped structure.

Facts:
• Approximate time of deployment: 1,5 hours for 100 meter;
• approximate costs of a 1 meter high barrier: €360/m';
• maximum standard protection height more than 3 meter.

Advantages:
• Easy to install;
• not location specific (each location do need a certain available width);
• possible to repair during service;
• easy to reuse;
• easy to construct corners;
• a defect of one part does not lead always to system failure.

Disadvantages:
• No long term solution;
• vulnerable for leakage;
• significant installation time;
• not suited well for large retaining heights;
• protection height is hard to increase;
• storage area necessary;
• small sharp object could still lead to major system failure;
• large site space required.

Air filled tubes
Air filled tubes are tubes with an long sleeve. The water pressure on top of the sleeve prevent the barrier of rolling over.

Facts:
• Approximate time of deployment: 1 hours for 100 meter;
• approximate costs of a 1 meter high barrier: €400/m';

5Blaauw et al. (2012) describes a maximum protection height of 9,75 meter which does not seems a preferable option to use
B.2. Structural design options

Figure B.35: Tigerdam example (retrieved from usfloodcontrol.com)

- maximum standard protection height more than 1,0 meter.

Advantages:
- Easy to install;
- not location specific (each location do need a certain available width);
- easy to clean and reuse.

Disadvantages:
- No long term solution;
- require relatively large width;
- vulnerable for leakage;
- significant installation time;
- not suited well for large retaining heights;
- protection height is hard to increase;
- storage area necessary;
- small sharp object could lead to major system failure;
- requires relatively flat surface.
Filled containers
Many systems are available at today’s marked with all kind of boxes or containers which can be filled with water or granular material to form a temporary flood defence system. The weight of the filled material provide enough stability. The granular filled containers (e.g. a street of bigbags joined together) are permeable and can only be used in an agricultural environment (it is hard to fill them with a material at, for instance, a city boulevard). Some types can be joined together and are made of an impermeable material. Figure B.37 illustrate the filled container type of solution (Ogunyoye et al. 2011).

Bigbags
A famous construction bag of 1 m³ is the Big bags. in this case, the are connected together to form a long stretch of fiber material containers which can be filled with earth or gravel. The have to be filled by loaders and can be positioned on top of each other to form a pyramid-shaped flood defence.

Facts:
- Approximate time of deployment: 1 - 2 hours for 100 meter;
- approximate costs of a 1,5 meter high barrier: €13,50/m³;
- maximum standard protection height to 2,25 meter.
Advantages:
- Relatively low visual impact;
- system height up to several metres;
- increasing height directly possible;
- installation by relatively unskilled labour;
- adapts to uneven terrain.

Disadvantages:
- Need to have a loading material nearby;
- need to have earth or gravel nearby;
- difficult to clean;
- can only be used a limited number of times;
- storage area necessary;
- long installation and mobilisation period;
- expensive to test;
- significant seepage may occur under the barrier.

\[ \text{Figure B.38: The bigbags flood defence system} \]

\textbf{Hesco barrier}

The Hesco system is a box defence barrier type consisting of wired meshes and geotextile connected together. The barrier has to be filled with earth or well-graded gravel by loaders. Figure B.39 shows the Hesco system.

Facts:
- Approximate time of deployment: 3 - 4 hours for 100 meter;
- approximate costs of a 1 meter high barrier: €50/m²;
- maximum standard protection height to 3 meter.

Advantages:
- Relatively low visual impact;
- system height up to several metres;
- increasing height directly possible;
- installation by relatively unskilled labour;
- adapts to uneven terrain.

Disadvantages:
- Need to have a loading material nearby;
- need to have earth or gravel nearby;
- difficult to clean;
• can only be used a limited number of times;
• storage area necessary;
• long installation and mobilisation period;
• expensive to test;
• significant seepage may occur under the barrier.

Figure B.39: Iraqi Army engineers fill a section of 1,20 meter Hesco bastions with a bucket loader (source: Hesco bastion Wikipedia)
Frame barrier
A temporary frame barrier basically consist of two parts, a support frame and a long sleeve made of impermeable membranes. Figure B.40 present an illustration of the frame barrier. The frames can be connected together, but this has to be done manually. The impermeable membrane extends at the river-side to form a long skirt providing stability and a seal with the ground surface (same idea as the flexible free-standing barrier).

![Diagram of frame barrier](image)

Figure B.40: Temporary solution; frame barrier

Geodesign Free standing barriers
This type of temporary free-standing system consist of supporting frames, plates and impermeable cover material like some geotextile types. By unfolding the frames and connecting them together the barrier is strong enough to retain the active water loads. Figure B.41 shows an illustration of the geodesign free-standing barrier. This type of system has bee used successfully for over a decade throughout many countries like Australia, Wales, Germany and Sweden.

Facts:
- Maximum standard protection height of 1,8 meter;

Advantages:
- Relatively low visual impact;
- adapt well to various terrains;
- can be placed on slightly uneven terrain;
- easy to clean and reuse.

Disadvantages:
- Very logistic / heavy transportation requirement;
- protection height is hard to increase;
- storage area necessary;
- long installation and mobilisation period;
- expensive to test;
- significant seepage may occur under the barrier;
- hard to unfold membrane during hard wind conditions.

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Free-standing barrier

The free-standing barrier is a impermeable material mostly made of a geo-membrane or plastic. (Ogunyoye et al. 2011) divides the barrier in two sub categories: Flexible and rigid barriers. the first kind of barriers are self supporting some without any hard elements. Water pressure should provide enough stability to hinder horizontal sliding and reduces seepage of the barrier. Figure B.42 shows the cross-section of a typical free-standing barrier. the water pressure helps the barrier to unfold. The rigid barriers have to be unfold manually and some types have a relatively long closure time.

Rapidam or megasecur

The Rapidam or Megasecur consists of elements that can be deployed by hand. As the water rises, the top skirt of the barrier will float. Water pressure inside the barrier prevent leakage and keep the upper skirt lifted. Figure B.43 shows the Rapidam installed and in use.

Facts:
- Approximate time of deployment: 12 minutes for 100 meter;

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B.2. Structural design options

- approximate costs of a 1 meter high barrier: €230/m²;
- maximum standard protection height of 2,0 meter.

Advantages:
- Minimal visual impact;
- quick and easy to install;
- no equipment required;
- small storage space required;
- easy transportable;
- easy to clean and reuse.

Disadvantages:
- Relatively high probability of leakage;
- protection height is hard to increase;
- testing only possible at higher water;
- significant seepage may occur under the barrier in uneven terrain;
- sensitive for winds, currents and overtopping.

---

**Boxbarrier**

The boxbarrier is a system consisting of linkable box-elements. After linking them together with joint members, one can fill them with water and cover them with lids. Figure B.44 is showing the boxbarrier system.

Facts:
- Approximate time of deployment: 1 hours for 1 meter height barrier, of a 100 meter stretch;
- approximate costs of a 1 meter high barrier: €205/m²;
- maximum standard protection height of 0,5 meter.

Advantages:
- Minimal visual impact;
- relative easy to repair during service;
- easy transportable;
- easy to clean and reuse.

Disadvantages:
- Storage area necessary;
- Limited height of elements;
- protection height is hard to increase;
- significant seepage may occur under the barrier in uneven terrain;

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Figure B.43: Rapidam, retrieved from Specialtyfabricsreview.com

Figure B.44: Boxbarrier system
B.2.4. House protection
Measures can be taken to protect households and buildings against floods. Protecting a house involve many small measures each with a probability of failure. At this moment it is very doubtful if these type of measures fulfil the current and new safety assessments. Despite this fact, there are occasions where one or multiple houses are positioned outside the ring-levee. Figure B.45 shows an example of measures to be taken in order to mitigate flood risks. The numbers represent measures to be taken in a common household. They include:

- Flood protection gate for doors;
- anti flood bricks;
- backwater valve;
- non-return sewer valve;
- brickwork and stonework protection.

Advantages:
- Tailor made solutions;
- cheaper with respect to a levee around one house;
- executing without large equipment.

Disadvantages:
- Many different measures;
- hard to calculate during a safety assessment;
- storage required;
- measures involve at someone’s else property.

B.2.5. Flood gates
These kind of barriers are big gates or doors which can be closed during high water. Many different types are available, depending on the material (often applied steel, aluminium or fibreglass), size and the way it rotates. In some cases the hinges of the door are at one side but there are doors which close vertically as a fabric door.

Facts:
- Approximate time of deployment: 15 sec to 3 minutes;
- approximate costs for 8 meter long, 1 meter high: € 25.000;
- maximum standard protection height to 2,4 meter.

Advantages:
- No external storage requirement;
- no transportation is required;
B.2. Structural design options

Figure B.45: House protection products, retrieved from UKfloodbarriers.co.uk

Figure B.46: Floodgate example

- system height up to several metres;
- system relative easy in deployment.

Disadvantages:
- Protection height hard to increase;
- possible to stuck during cold weather conditions;
- often always visible;
- sensitive for possible leakage at the sill;
- sensitive for settlements.

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Toolbox to form realistic alternatives

This appendix formulate the procedure used to define, design and screen the levee design alternatives for both case studies. The procedure forms realistic alternatives which will be analysed further on their LCC.

This appendix consist of three main parts of the procedure:

- Defining alternatives (paragraph C.1);
- designing criteria (paragraph C.2);
- screening criteria (paragraph C.3).

C.1. Defining alternatives

This chapter set out the way how levee design alternatives will be defined during the research. One import aspect regarding defining alternatives is the search to a wide variety of possibilities in order to consider "out of the box" alternatives too.

The first paragraph starts by explaining how an alternative can be formed. Paragraph C.1.2 evaluate different levee alignment options followed by the requirements during the forming of these options.

C.1.1. Alternative organisation

The purpose is to define a wide range of possibilities, or alternatives, which might be used in a flood defence design project. Solutions for flood hazards are categorized to identify the area the alternatives can be used. for instance, agriculture areas are often defended by green alternatives\(^1\) while urban areas often struggle with limited space leading to a more structural solution. Besides levee type solutions, also measures to adjust the river discharge profile can be though of in order to increase the safety. Those measures should be looked at when considering the whole river system, rather than just investigating one ring-levee or an element of it. Therefore, measures like deepening or widening the river profile are not elaborated as alternatives during this research.

A flood hazard can be prevented by different kinds of flood management systems. One concept is Multi-layered Safety or MLS\(^2\) (Hoss 2010) which divides measures that potentially degrade the impact of flooding into three layers: Prevention, Spatial planning and Crisis management. All alternative within this research are Prevention measures only.

Figure C.1 shows how the alternatives are subdivided. Besides different structure types, like green

\(^1\)The Netherlands describes a green alternative as an earthy levee with mainly a grass cover which is one of the most applied solutions in the country.

\(^2\)Dutch: Meerlaagseveiligheid, MLV
solutions, also the levee alignment is part of an alternative. The total process of defining alternatives is treated in paragraph 2.1.

![Flood defence system flow chart](image)

A flood defence system should be analysed over its whole ring-levee. This include the possibility to change the levee alignment. For instance, a levee alignment on a higher surface level which might reduce project costs. Together with the determined levee alignment option(s) and a the green- and/or structural option(s), an alternative is formed (see figure C.1). The "u" represent "or/and" as a levee structure can consist of both a green and structural option.

Different agencies and companies are dividing the structural solutions in temporary and demountable systems (Ogunyoye et al. 2011, van Heereveld 2008, Peijnenborgh et al. 2006). Some solutions (e.g. Aquabarrier) differ in groups over the literature which embraces the idea that there is no hard restrictions between the categories. In this case a subdivision is chosen in three groups regarding the structural options: Permanent, demountable and temporary systems.

### C.1.2. Options of levee alignment

Most levees along the river Meuse in Limburg are constructed in the few years after 1995. During these years levees were constructed through peoples backyard, at boulevards and along existing roads with the attention to stay there for 10 to 15 years like mentioned in chapter 1. Today’s levee designs have typically lifetimes of 50 to 100 years. Hence, some basic assumptions are different. A new location of the levees might be a better solution.

At some locations there is a possibility to move the levee further landward. It might lowers the project costs because less material can be used due to higher grounds or cheaper alternatives due to better ground usage. There might be solutions thought of which combine on one side of the river the levees will be moved landwards while the other side, in urban areas, the levee can be moved to the river-side.

There are examples of levee changes in urban areas such as Kampen, the Netherlands wherein the levee alignment coincide present buildings. Here they used houses and the old city wall as multifunctional flood defence. By constructing seepage decreasing walls, anti-flood gates the area has been made flood-proof. At some locations the masonry walls are improved and streets are blocked with demountable or/and integrated systems (WGS 2007). Figure C.2 shows part of a map of Kampen

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C.1. Defining alternatives

Figure C.2: Example of a levee alignment options at Kampen. In red, the flood defence running true houses and the city wall. Top blue part of the figure is the river IJssel (WGS 2007).

including the drawn flood defence location. The levee is moved landward where 80 houses are out of the ring-levee and still vulnerable to flood water.

Before beginning with summing up all possible alternatives often the problem is clearly stated. After defining the problems one can think of all different locations of the levee such that one or more problems might be solved. Of course one has to consider in all circumstances the same alignment as the present situation. Figure C.3 illustrate 4 levee alignment options of an imaginary urban area. The first option, number 0 present the same alignment.

To create these alignments several aspects can be considered: Higher grounds reduced the total volume of the levees which could be beneficial for the total costs of the levee. The use of other structures like houses, parks or roads are also a possibility (option 2 and 3). Sometimes, on first hand, it is not allowed to move or extend the levee at riverside as it decreases the flow profile of the river. Still, this might be a solution if one is able to compensate with an levee alignment more landward nearby. These kind of options are more difficult to test as it has to be checked with hydraulic river models if the results in negative aspect further up- or downstream. A more indirect aspect is the environmental condition which can be increase by changing the levee alignment. The importance of these aspects partly depends on the stakeholders preferences which makes search for these option more difficult. Therefore, it is preferred to define these options thinking about each stakeholders view of the levee system (An example is a deteriorated road which has to be upgraded within a few years that can be increased as levee). If not all preferred options are analysed during the funneling process, extra doubts might occur by other stakeholder when discussing the results. It could harm the

3As this options does not actually differs from the present situation, the number 0 is used.

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trust of the design team / levee management.

Normally, one consider the whole ring-levee to identify then levee alignment options in combination with other ring-levees, future development of land-usage and the river profile. This is often done by intergovernmental programs like the HWBP.

In a later stadium sub-sections can be analysed each with a different option. If one consider to allocate preferable options per sub-section in this ring-levee part, the number of options increases which probably is a disadvantage. To continue this part with the illustrated example figure C.4 indicate a ring-levee part subdivided into 3 section. Each section has its "own" option as solution. Hence, sub-section A should be analysed for options 0 to 3 as well as sub-section B (inclusive the consideration the levees should be connected to each other in total. In contrast to this The bottom part of option 0 and 1 are actually the same what is in contrast to the described manner of pinpointing levee alignment options (figure C.4). This illustrate the difficulty of forming different alignment options. Option 1 can be included due to future planning of the ground (e.g. as a boulevard or recreation area), prospect to archaeological finds or just to search a cheaper alternative. At the end it doesn’t matter how the options are considered as long as the reason and the difference between these options are clear.

![Figure C.4: Illustration of the formation of an levee design alternative by combining various levee alignments](image)

**C.1.3. Requirements during levee alignment analysis**

In order to create a fair analysis, different constrictions have to be defined. A stakeholder preferred situation should be considered somehow. Therefore, a quick and dirty stakeholder analysis has been carried in order to get an understanding what preferences are important. In general this type of analysis only take a few minutes to a few hours of time (pinpointing all stakeholders and list their interest, concerns and preferences).

The first rather clear example is to define in all cases the option 0; present situation. This is the favourable option regarding to permits and a good middle ground with respect to all stakeholders.

There should be at least one option which does not influence the river flow profile negatively. The river manager is an important stakeholder who strongly advice to not decrease the river profile during levee strengthening projects.

A more complicated constriction is to consider at least one option in which the prospected LCC are the smallest. Expert judgement is the only way to investigate which option could end up as the cheapest in this stage of the alternative analysis.

At the end, these criteria could end up with only one option. If no other stakeholders have important aspects to include during this step of the analysis, one can continue with defining alternatives with one alignment option.

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C.2. Designing alternatives

Design criteria are formed with the aid of various guidelines and rules of thumb. A summary of these criteria are shown below. A distinction is made between criteria regarding the function of the levee, the dimensions of the levee, its strength and the requirements during maintenance. The framed boxes are the actual design criteria per aspect.

C.2.1. Levee functions

Flood risk reduction
The prior function of the levee is to reduce flood risks. This flood risk correlates with a certain probability of exceedance/probability of a flood.

Currently, the water board is planning and executing the levee system along the river Meuse in Limburg such that they fulfill the present statutory assessment, the annual probability of exceedance of 1/250. After a period of approximately 50 years, these levees need to be increased and/or reinforced, both due to the implementation of the new safety approach, and the expected increase of river discharge. Unfortunately, at the moment the Dutch government is still processing the translation of the new safety approach into a practical design and check methodology. Hence, assumptions have to be made regarding the new safety approach.

The first assumption enables to "use" the upcoming safety approach:

Levee alternatives will be designed based on the present safety approach.

The above assumption entails that, a future annual flood probability of 1/1250 will be designed with the present statutory assessment, and a water level with an annual probability of exceedance of 1/1250.

Furthermore the flood probability according to the new safety approach of Dutch levee-systems itself is under development. Berkhof et al. (2013) gives insight in these flood probabilities. Here they examined both ring-levees and advised an probability of a flood of 1/1250. This lead us to the following assumption:

Design has to fulfill a minimum protection level corresponding with an annual probability of exceedance of 1/250 and an annual probability of exceedance of 1/1250 after major maintenance.

Extreme large discharges are unlikely to occur with the same chances as extrapolated date of the discharge-probability relations. It is physically very unlikely to have discharges greater than 4.600 m$^3$/s at the river Meuse (Berkhof et al. 2013). With these values many areas of Belgium will be flooded. Hence, a physical maximum discharge should be assigned to the river Meuse which limits future benefits (chapter 3). This lead us to the following design criteria:

A physical maximum discharge is assigned of 4.600 m$^3$/s for the river Meuse in Limburg.

Infrastructural elements
A levee often affect present infrastructure like roads positioned on top of a levee or a sewer system which lays within a danger zone. There are situations thinkable where the location of a road changes which result in a smaller levee design. Another example is the reorganisation of a park to increase recreation in combination with establishing a proper levee system. The following criteria is assumed to be required:

All existing infrastructural functions in, on top and along the levee have to be preserved.

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River flow profile
Flood defence measures are not allowed to have negative hydraulic influence of the river. This requirement is an important aspect during the design of the alternatives. The hydraulic impact into the river flow profile will a screening criteria. However, the alternatives are assumed to be only realistic if the reduction of the flow profile will be compensated. A reduction could come from alternatives who are positioned at the river side of the present situation. This assumptions lead to the following design criteria:

The decreased area of the river flow profile due to a change in levee alternative, should be compensated with the same area somewhere in the same cross-section.

The adjustments are not in line with the nowadays strategy of first defining places where more space can be given to the river and secondly the reinforcement of levees. Perhaps at the end of the screening process, this type of alternatives are rejected just because of this reason. Therefore, it is important to start the dialogue on time to discuss these type of alternatives.

In contrast, reinforcing a levee at its land side might conflict with structures and land-owners. There will always be struggle finding a balance between the provided space, preferred space for an optimal design and the costs of different levee alternatives.

Monumental buildings
Some areas along the river Meus in Arcen have historical buildings like the historical city wall of Venlo and often of important cultural value. Therefore, a design criteria is assumed:

Monumental buildings and structures have to be preserved;

Floodable levees
An ongoing discussion is present over the past few months/years which concerning the requirements to have floodable levees along the river Meuse in Limburg. The basic idea was that, the change to primary levees should not enhance big disadvantages further downstream of the river. The levees had should have a maximum height up to the annual probability of exceedance of /1250.

However, this requirement increases the difficulty of designing levees. There is a strong belief that this requirement will be cancelled for the primary levees in the nearby future. Therefore no attention is given to floodable levees.

C.2.2. Levee dimensions

Levee height
Approximately 95% of the levees in the northern part of the river Meuse in Limburg failed the statutory check\(^4\). In many cases a levee fails the check of the mechanism of overflow and overtopping (de Groot 2012). When determining the height, overtopping (and overflow) is taken into account together with an additional water height and the local reduction of the levee height in contrast to the reference level. The VTV (2007) described to have a minimum crest height with respect to the reference level of:

\[
h_{cr} \geq \text{reference level} + \text{additional increase water level} + \text{freeboard}
\]  

(C.1)

Climate changes causes larger discharges resulting in higher water levels regarding a certain probability of exceedance. Secondly, squalls, gusts, oscillations and local surges have to be accounted creating a higher water level. Furthermore, ground action like settlements decrease the crest height over time as well. The standard determination of the design height is presented in figure C.5.

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\(^4\)This was the 3\(^{nd}\) delayed statutory check of the Netherlands, but the first check during the lifetime of the levees along the Meuse in Limburg
Designing alternatives

For normal levees, at least a minimum crest height have to be applied of the reference level + 0,8 meter. An exception is made for levees along the Meuse (ENW 2007). There they estimate a crest freeboard of 0,50 meter during the alternative analysis in line with the nowadays used freeboard of WPM source: WPM.

The freeboard is checked with the HYDRA-R model in Arcen. The required crest height is calculated belonging to a overtopping discharge of 1 m$^3$/s/m and 0,1 m$^3$/s/m. The resulting freeboards (not taking in account ground action) are 0,34 and 0,54 meter respectively. When one neglects ground action, these heights coincide with the 0,50 meter of the water board.

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<td>[m N.A.P.]</td>
</tr>
</tbody>
</table>

The ground action (Settlements, compaction and local ground subsidence) are much smaller in Limburg in contrast to the Western part of the Netherlands. No ground subsidence is suspected in the Meuse-valley in Limburg according to Berkhof et al. (2013) towards 2100. At this moment, the water board neglects ground action under normal circumstances. The compaction will (partly) be solved during construction.

From the above stated aspects, the following assumptions is made:

Crest freeboard of 0,50 meter.

Next some attention is given to the height of the water levels at the start of the levee construction and during major maintenance.
The hydraulic boundary conditions have been formed deterministically and probabilistically. A flood with an annual probability of exceedance of $1/250$ is the design safety level. The levee will be increased/reinforced after a period of approximately 50 years. As the base time (the year of completion) is in the year 2020, the levee should fulfil the hydraulic boundary conditions of 2070. The new safety approach require an annual flood probability of $1/1250$ (Berkhof et al. 2013).

The hydraulic boundary conditions are stated in Appendix D. Within this appendix, several assumptions c.q. estimates are formulated. For instance, the relation between discharge and the water levels are extrapolated. The exact water levels described within this report could differ from the actual formed levels at the moment these projects are executed.

**crest width**

A minimum crest width of 3 meter is required for green earthy levees according to the TAW (1994) and ENW (2007). The main purpose is to have enough space for inspection, maintenance and the accessibility during high water. This minimum crest width does also apply for parallel levees.

The Water board Peel and Maasvallei has additional preferences concerning the crest width in order to make the inspection and maintenance of a levee easier. They propose a minimum crest width of 4.5 meter such that an inspection path of 3 meter can be applied. This lead us to the following design criteria:

| A minimum crest width of 4.5 meter is required unless the maintenance and inspection path is located somewhere else. In those cases the minimum required crest width becomes 3.0 meter. |

**Slope angles**

Often it is advised to use a slope of 1:3 as a practical fist estimate for inner as well as outer slope. A more gentler slope is favourable regarding the macro stability but requires more space. A steep slope also reduces the ability to inspect and maintain the levee easily. A more gentler slope might decrease the discharge profile if one has to construct the levee into the riverside (ENW 2008).

| Slopes have a minimum steepness of 1:3 unless structural elements are present limiting macro-instability. |

**View historical city-centre**

According to Berkhof et al. (2013), levees part of a historical city center should have a maximum of 0.80 meters with respect to the direct surface. Although the primary function of the report is to draw up a preferred vision in the Netherlands rather than giving explicit requirements, it does give some understanding of the importance. People driving, bicycling and walking along boulevards should be able to watch the river.

During a final design stage one could change this height depending on the preferences of the stakeholders. for now, it is assumed to be the maximum height during the execution phase. As both case studies involve historical cities/villages it leads to the following design criteria:

| A maximum structure height of 0.80 is allowed with respect to the surface in order to preserve the ability to have a proper view on the river. |

Of course there is always the ability to raise the surface or to change the levee alignment.

**C.2.3. Levee strength**

**Clay cover**

In most cases, clay is used underneath a cover-layer for green alternatives. The clay prevent erosion,
piping, instability and is less permeable than other types of earth such as sand. In the Netherlands, clay is categorized in three types of erosion resistance; 1, 2 and 3 where type one offers most resistance followed by two and three. Currently, the water board apply a 1,0 meter thick layer of clay on the slope and berm on the river-side and the cover of a green levee. This thickness is used as design criteria:

A clay layer with erosion type 2 and a thickness of 1,0 meter will be applied on the crest and slope/berm on the river-side of a green levee.

Several ongoing projects of the water board include a clay layer at the river-side to decrease the probability of failure due to piping. An extra clay layer of 5 meter and 1,0 meter thick is assumed to be designed starting from the tone of the levee at the river-side.

A 5,0 meter long clay layer of 1,0 meter thick is required to decrease the probability of failure due to piping unless a structural element like a sheet pile is present.

**wall system**

Commonly two wall-systems are used within a levee; a steel wall (sheet pile or combi-wall) and a diaphragm-wall. The sheet pile is the most favourable one in terms of costs (assuming standard execution ability). To elaborate a first sketch design the following criteria is formed:

Sheet pile wall will be used for all retaining wall types with optional a concrete capping beam.

The concrete capping beam can be raised during major maintenance. Other levee design alternatives include a demountable-, dynamic- or a glass-wall system which is more difficult to reinforce/increase.

A rule of thumb suggest the approximate length of a cantilever wall to be three times the retaining height in order to maintain enough stability. The sheet pile length can be reduced by including anchors. However, besides retaining ground, the sheet pile limits piping, too. Therefore the rule of thumb regarding the sheet pile length is used as design criteria.

The total length of a cantilever wall is three times its retaining height.

**C.2.4. Levee maintenance**

A levee requires a path to maintain and inspect properly. In the most ideal situation the path is positioned on top of the levee. The path is available during an extreme high water and one is able to inspect both sides relatively easy. When it comes to structural levee alternatives, the preferred location of the maintenance and inspection pat is at the land-side. The following design criteria is formed:

The preferred maintenance and inspection path position is on top of a green levee or at the land-side of a structural levee alternative.

Subsequently the design width of the path is assumed to be 3,0 meters which is in line of the water board preferences.

The maintenance and inspection path has a width of 3,0 meters.

Although the preferred location of the maintenance and inspection path is on top or at the land-side of the levee, there are options thinkable where this is not the case. However, the importance of a proper path during extreme high water events for demountable- and dynamic systems is much larger. Therefore, an additional design criteria is defined:

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A maintenance and inspection path has to be on the accessible during the governing high water event.

The maximum distance between a structural levee and the maintenance and inspection path have to be maximised. During the sketch design, the following assumption is made:

the maximum distance between the maintenance and inspection path and a structural alternatives is 2,0 meters.

Besides the location and width of the path, a minimum required passage height should be assigned, too. A demountable system underneath a bridge or building could enhance extra risks. The locations where height becomes a problem are very rare within a levee design and should always considered with great care. Besides the ability to maintain the levee properly, there are other factors important as well. For instance, temporary closure of a pass-way during extreme high water conditions could limit the number of evacuations routes. Besides that, there are many different types and structures and often tailor-made. Therefore, no design criteria as it is hard to define one properly.
C.3. Screening criteria

This paragraph states the criteria which will be used during the screening process. One searched for the most significant criteria influencing whether an alternative is realistic and cost effective. The cost effectiveness is taken into account during the LCCA in chapter 3. Whenever a levee design alternative is realistic is evaluated here via the screening criteria.

The ideal alternative fulfills all design- and screening criteria, is robust, cost effective, and satisfies all stakeholders. The most unwanted alternative in this stage of the design procedure fulfill all design criteria, but is terrible in terms of robustness, cost effectiveness and is unwanted by all stakeholders. One seeks for a realistic alternative which converge as most as possible to an idealistic alternative, although some unwanted aspects are still present.

The following screening criteria are described in the upcoming sub-paragraphs and often divided by multiple aspects:

- Constructibility;
- Hindrance during execution;
- Hindrance during usage;
- Hydraulic impact;
- Nature;
- Monuments;
- Trees
- Maintenance and inspection;
- Connection between ring-levee sections;
- Project investments.

The order of the above criteria is randomly. Per criteria points are specified per aspect (one criteria contains one or more aspects) in order to combine these aspects into one screening criteria. By summing up these points a certain score can be formed which is assigned to a “+” (good), “0” (doubtful) or “−” (bad).

The relative difference between alternatives is more important rather than the score itself. There probably are cases in where all alternatives score good, doubtful or bad, respectively. In both ways, there is no relative difference between those alternatives.

Per aspect scores are estimated by judging whether or not an alternative is realistic. At the end one or more examples are formed in where a levee design alternatives stays realistic, becomes doubtful or is bad.

Often part of the aspect apply in the levee section which reduces the score. Hence, if an aspect is not preferred, but is very small in relation to the whole levee section, their input within the screening criteria should be limited. In these cases the gathered points per aspect should be multiplied by a factor $\zeta$ as follows:

$$\zeta = \frac{\text{length aspect}}{\text{total length levee section}}$$  \hspace{1cm} (C.2)

The scores, assigned per screening criteria, are mostly based on expert judgement.

The sum of the points in all aspects within one screening criteria, should be evaluated via the scores:

- Total score is 30 ore less;
- Total score is between 31 and 60;
- Total score is 61 or higher.
Each project involves different stakeholders, different interests and a variability in importance of the screening criteria. Hence, screening levee design alternatives is a place-specific procedure. In some cases a tree avenue can be highly important while at another place, the same trees much less. Therefore, levee managers in cooperation with other stakeholders do prefer to include some priority level within the screening phase. This can be done by ascribe an X number of points to each screening criteria together with the governing stakeholders.

C.3.1. Constructibility
The constructibility screening criteria identify obstacles during execution which influence whether an alternative is realistic or not. This screening criteria is subdivided into 4 aspects:

- Construction difficulty;
- Construction risks;
- Accessibility of equipment;
- External factors.

Construction difficulty
Having an alternative with no or minor construction difficulties is favourable comparing to alternatives where the construction phase is more difficult. A difference is made between the construction difficulty (e.g., it is easier to execute a sheet pile vertically than piling it in a different angle) the construction risks (e.g., the possibility the sheet pile result in unwanted building settlements). The construction risk and subsequently the accessibility of equipment and external factors are formulated later. Here, the construction difficulty for each construction element is described separately.

A sheet pile, or diaphragm wall, is an often applied structure element when it comes to levee designs. Besides retaining ground or water it is able to prevent piping. Scoring the constructibility of a sheet pile is based on the angle, length and the ability to pile with the given equipment. The actual length of a wall enforce to use bigger equipment which is more expensive and more difficult to use. The total length is categorized in three sizes. Each size coincide with a certain number of points which should be added to the total score of constructibility.

- 4 Points: < 10 meter of sheet pile;
- 7 points: 10 - 20 meter of sheet pile;
- 10 points: >20 meter of sheet pile.

Piling a sheet pile under an angle rather than vertically encompass more difficulty and hence, is less preferable. Therefore, when a levee design alternative include a sheet pile at an angle, 10 points extra should be assigned to the total score.

- 10 points: Sheet pile not vertical.

A sheet pile can be piled from water via pontoons or from land. In the preferable situation only one of these two methods can be used for all sheet piles. When a levee alignment option enforces to use both methods, 10 extra points are assigned.

- 10 Points: Piling from water and land.

Having a steep slope and no other place to pile, another 10 points can be added due to extra measures during the execution.

- 10 Points: extra measures during the execution of a sheet pile.

Anchors differ in construction difficulties too. Whenever anchors are necessary, 5 points should be added to the total score. The easiest way to construct anchors is above ground (water) level with an anchor plate (or present situation). Whenever applying anchors at an angle, 5 extra points are assumed to add to the total score. Applying anchors at an angle is still a standard procedure and therefore appointed with a low number of points. Furthermore there are many different types of anchors, often related to a certain contractor. It is very hard to assign a difference between anchors as each contractor assigns their preferred types differently. Whenever one really can identify a extra difficulty regarding the construction phase, there is the ability to include extra points if argued well.
C.3. Screening criteria

- 5 Points: Anchors are necessary;
- 5 points: Applying anchors at an angle.

A water defence consisting of concrete elements with equal dimensions over its full length can often be seen as beneficial. Besides the effective use of formwork and falsework, there is a relative large repetition factor. If there is a concrete element present, the following points should be taken into account:

- 5 Points: Insitu reinforced concrete floor;
- 10 points: Insitu reinforced concrete wall;
- 5 points: Prefab reinforced concrete element.

Poring concrete with large height increase difficulty during the construction phase. Besides the fact that formwork and falswork have to be strong enough to resist the concrete pressure, the concrete has to be applied in multiply layers in order to prevent possible de-mixing. This lead us to the following points:

- 0 Points: Insitu reinforced concrete smaller than 1 meter;
- 5 points: Insitu reinforced concrete between 1 and 2 meters;
- 10 point: Insitu reinforced concrete larger than 2 meter.

Levee structures sometimes have to be provided with a pile foundation due to a lack of barring capacity, or to increase structural stability. The piles enhance additional machinery and extra transport which increases difficulty to construct. Therefore an additional 10 points should be taken into account when a pile foundation (both concrete or steel piles) has to be applied.

- 10 points: Require a pile foundation;

Construction risks

Placing a diaphragm wall or sheet pile nearby other buildings entails a certain construction risk depending on the soil type, depth and foundation type of the existing building. Therefore points are assigned by considering the distance of the “influence line” and the building via the following equation:

\[ D = L_{\text{distance}} - L_{\text{influence}} = L_{\text{distance}} - h_{\text{wall}} \tan(45 - \phi/2) \] (C.3)

Where:

- 15 points: D < - 5 meters;
- 10 points: -5 < D < 0 meters ;
- 5 points: 0 < D < 5 meters;
- 0 points: 5 < D meters.

Often anchors have to be applied which can affect nearby buildings also. An anchor entail less risk as a sheet pile. The same influence length can be calculated from the outer end of the anchor with equation C.3, but with other points:

- 15 points: D < 0 meters;
- 5 points: 0 < D < 3 meters;
- 0 points: 3 < D meters.

Some levee design alternatives are much more sensitive for settlements as others depending on the weight of the structure, foundation and soil parameters. The main category of levee types who are sensitive for settlements and ground subsidence are the demountable and dynamic systems. Therefore an additional 5 points are assigned to these type of systems.

Generally the construction risk increases when it is necessary to decrease the ground water level. Therefore an additional 5 points are assigned to alternatives where a drainage system is required.

It is very unlikely that a flood of the river Meuse occurs in the summer. Levee reinforcement are planned within the dry season to limit the possibility to have a flood during the execution. Even though, there is a possible risk that such extreme event happen within the execution period. An extra 5 points are assigned to levee design alternatives in where there is temporary no flood defence during the execution per month.

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• 5 Points: Sensitive for settlements;
• 5 points: Likely to decrease ground water level;
• 5 points/month: present levee structure has to be taken out.

**Accessibility equipment**

During the execution phase there are many aspects influencing the constructibility. Some levee design alternatives require large pontoons, barges, hydraulic excavators, poring equipment, etc. Levee design alternatives which require a large amount of equipment commonly enhance a larger difficulty during the execution, too. Therefore the following points are assigned:

• 5 Points: Land equipment has to be transported over water;
• 5 points: Weekly transport is hard to access project (e.g. narrow streets, limited passing height, maximized weight or prohibition of large vehicles);
• 5 points: there is insufficient storage place;
• 2 points: execution required at both sides of the river.

**External factors**

In some cases the constructibility depends on external factors. Very cold or warm weather conditions are not perfect for the drying process of the concrete. Or a high precipitation could raise the ground level unwanted high. The following external factors are given with the assigned points:

• 3 Points: Sensitive for large precipitation;
• 3 points: Sensitive for very warm conditions;
• 3 points: Sensitive for large wind;
• 2 points: Sensitive for very cold conditions.

**C.3.2. Hindrance during execution**

Hindrance during the execution phase always involve unpleasant situations like the inaccessibility of building or the temporary decrease of available parking lots. There is a difference between hindrance during the usage of the flood barrier and hindrance during the execution phase as the latter one is only temporary. Missing a parking lot for just a few weeks is much different than losing your parking lot for many decades. This screening criteria state the hindrance during the execution phase.

During execution hindrance will be encountered by residents, companies, traffic (cars, bicycles, pedestrian and ships), recreation and during emergencies (like a fire, a major traffic accident or a flood). To include a difference in importance between these aspects, point are defined which result in a good, doubtful or bad score after summing them up. However, the relative importance of temporary hindrance comparing to other criteria of the screening procedure, is questionable. Does one really want to chose an alternative due to a hindrance of just few weeks?

Several "extreme conditions" have to be defined to qualify under what conditions the hindrance during execution screening is good, bad or doubtful. Losing a parking lot for just a month should not be assigned as bad nor doubtful. But having a building pit for several months in front of a primary school for autistic children may make a significant influence of the alternative analysis. Of course such extreme conditions are rather unique and hard to elaborate in a standardized manner. In the majority of these cases the hindrance during execution will be assigned as good. The best method, if one prefer to investigate the execution hindrance, is to analyse if there could be any extreme condition in general. if not, one may continue with the subsequent aspects within this screening criteria. Some extreme conditions are give as example what may result in a bad score:

• One is unable to leave a school building properly during emergencies;
• a decrease in navigation ability such that a vessel has to wait more than 10 minutes on average;
• Stores and companies are unable to transport their daily products (e.g. a supermarket which supplement groceries with a truck).

Besides these extreme conditions, some general aspects regarding the hindrance during execution are evaluated below:

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**Residents**
The following points apply for hindrance regarding residents if it affects at least 10% of the residents living along the flood defence in the section.

- 5 Points: Temporary decrease in available parking lots;
- 10 points: Temporary hard to enter and leave one's house;
- 5 points: Temporary limited access in front or backyard;
- 5 points: Large sound hindrance;
- 5 points: Dust hindrance.

**Companies**
A construction site nearby stores may lead to a (temporary) decrease in costumers. Good agreements should be made and solutions should searched for to limit the hindrance. The following points are assigned:

- 5 Points: Hard to enter and leave a company building with daily costumers;
- 5 points: limited parking lots;
- 5 points: change of the intern emergency route;
- 5 points: difficulty to enter or leave building by emergency services.

**Traffic**
Roads and paths are often temporary closed during the execution phase. Aspects like the duration of the (partly) closure of this hindrance is one example which can be used within the screening analysis. Points are assigned to the hindrance of traffic as follows:

- 10 Points: Temporary road closure per month;
- 5 points: Temporary bicycle/walking path closure per month;
- 10 points: Temporary emergency path closure per month;
- 20 points: Temporary and partly closure of then navigable waterway per month.

**C.3.3. Hindrance during usage**
Occasionally, a new flood defence system cause unwanted hindrance due to its presents closed or open. the following aspects are chosen within this screening criteria:

- Accessibility;
- available space left;
- privacy;
- safety.

**Accessibility**
Infrastructure, houses, public buildings, stores can all be influenced by the new levee design. Whether or not people have to struggle to access their homes, gardens, etc. should be included into the screening criteria. Each indention below describe an accessibility aspect and their scoring.

Within some alternatives residents loses their back entrance. This aspects is a major influence and should be assigned as bad if it happen to the whole section along the ring-levee. An estimated maximum of 50% of the whole section which loses their back entrance is still assumed to be realistic (30 points). The latter only holds if all other aspects within the screening criteria are zero. With this 50% and the defined $\zeta$ in equation C.2 leads to C.4.

\[
Score\ back\ entrance = 60 * \zeta \quad \text{(C.4)}
\]

There are some examples in where the levee structure runs through peoples backyard. If a levee design alternative change the backyard in such a way that people need to walk around or have to use stairs to entrance it, one should include this aspect in the screening criteria. if the walk is just within 5 seconds (or 5 steps of a stair), it is assumed to be negligible. Furthermore the same 50% limit is used as before but with half of the points assigned(see equation C.5).

\[
Score\ stairs = 30 * \zeta \quad \text{(C.5)}
\]
If there are other functions or building at the other side of a road or path which are easy accessible by residents, but not due to an levee design alternative, points should be assigned as well. A heightened road over the full length of the ring-levee section is assigned with 25 points (equation C.6).

\[ \text{accessibility crossing roads} = 25 \times \zeta \]  
(C.6)

Besides the daily use of our infrastructure and houses, there are aspects which should be considered during extreme high water conditions, too. A flood forecast activate the closure procedure of a dynamic or demountable system. Having a potential flood and a hindrance during the evacuation may result in a larger amount of victims. This evacuation management depends on several things entailing a large effort to come up with realistic numbers\(^5\). To limit analysing time, two aspects are identified to score the accessibility during a flood; the possibility to enter or leave a building and the usage of an (evacuation) road.

The inaccessibility of buildings, is not of the biggest concern. A good crisis management limit the risk of people trapped in their houses. If a system is chosen to defence the houses, people may reconsider why they should evacuate as the system should prevent for flooding the buildings. The assigned score is assumed to be still doubtful (60 points) when 100% of the buildings along the ring-levee sections are inaccessible.

\[ \text{Accessibility buildings during a flood} = 60 \times \zeta \]  
(C.7)

If a evacuation path or road is blocked, the difficulty to assign a proper score is even larger. It depends mostly on the number of evacuation paths, the time to evacuate, the width of each road, whenever cars are nearby, etc. There is a big difference between roads whether they are positioned perpendicular or in longitudinal direction along the river, whether a road going to higher grounds or not and if it goes through unprotected areas. Therefore an assumption is made that, if a certain road-type is not accessible due to a flood, these scores should be added to the rest:

- 10 Points: Walking and bicycling path;
- 20 points: Maintenance or inspection path;
- 20 points: One way access road;
- 25 points: Two way access road;
- 30 points: Distributor roads or bigger.

During the scoring procedure, only one of these can be chosen to prevent large scores given just due to the decrease in road usage during a flood.

**available space left**

There are alternatives and areas thinkable in where infrastructure or building can be expanded due to that levee design. Expanding a house within your backyard is just one example where a levee could have impact. Equation C.8 include the ability to expand. One should only consider these cases which differ with respect to the present situation. After all, if a road can't be expanded beforehand, it is not fair to give levees with the same alignment as the present situation a bad score. Furthermore only the highest number of the constant should be applied as the total importance of this aspect within the hindrance during usage screening criteria has to be limited.

\[ \text{Available space left} = C_i \times \zeta \]  
(C.8)

\( C_i \) is the constant which can be chosen from the list below:

- 20 points: Can't expand building(s) (only if it entails their own property);
- 30 points: Can't expand roads;
- 5 points: Can't expand bicycle or walking path;
- 200 points: Can't expand rail track;
- 100 points: Can't expand flood defence.

\(^5\)Analysing a crisis management per alternative or deriving rules of thumb are time consuming and is a thesis subject at its own.

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Privacy

Functions along a levee can change after realisation of a project. An increase in recreation activity for instance may result in a decrease of the privacy of residents. The privacy part of the usage hindrance doesn’t seem important at first hand but is a real struggle during ongoing project like the current levee reinforcement in Mook by the water board.

To come up with a proper score for privacy, one could relate the ability to watch somewhere inside with the frequency of traffic passing by. Moreover this can be related to the distance from where the traffic is in relation to the buildings. One has more privacy if a bicycle path is 100 meters away instead of 20. The last part of the score involve the relative size of this aspect related to the ring-levee section. The score should be calculated with respect to the present situation.

An increase in frequency can be assigned with a value of 1, 2, 3 or 4 in where it represent the factor of increase (for example, a walking path is forecasted to be used 3 times more often after the realisation phase, it will be assigned as 3). 4 is the maximum score in order to limit the impact of this aspect into the total screening criteria.

A green levee might be raised which enables to have a better view in someone’s else house, backyard, etc. In other words, the ability is increased ending up with a decrease in privacy. The ability is assigned with the values 1, 2, 3 or 4.

A increase in distance lead to an increase in privacy. One is able to divide the score by the distance to create a total score. The distances are categorized starting with:

- <10 Meter (the road/path lays within 10 meters of the houses/backyards);
- 20 meters (the road/path lays between 10 and 20 meters);
- 30 meters;
- ... ;
- 90 meters;
- >100 meters (maximum estimated distance in where a road/path could have privacy influence).

Equation C.9 shows the relation between the frequency, ability and distance which lead to a certain privacy score multiplied with a constant and the factor $\zeta$.

$$C = \frac{frequency \times ability}{distance} \times \zeta$$  \hspace{1cm} (C.9)

The constant is added to adjust the formula such that a worst case scenario is equalized up to the importance of the privacy in relation to the other aspects in the screening criteria. The estimated maximum due to the privacy is assigned to 60 points which result in a constant $C \approx 75$.

Safety

Some levee design alternatives entailing an unwanted drop of the safety. A slope in combination with a vertical wall on someone’s property or an obstacle nearby a road are just two examples of a decrease in safety. The following points are assigned:

- 15 Points: Obstacle within a road;
- 5 points: Obstacle with maximum 2 meter nearby a road
- 6 points: Obstacle within a bicycle path;
- 2 points: Obstacle with maximum 1 meter nearby a bicycle path;
- 3 points: Obstacle within a road during closure of the flood defence system;
- 1 point: Obstacle within a bicycle path during closure of the flood defence system;
- 20 points: Dangerous situation in the backyard;
- 10 points: Dangerous situation in the front-yard.

C.3.4. Hydraulic impact to future river widening projects

An important aspect is the hydraulic impact of the river due to the levee design. A change in river flow profile should be investigated as a (significant) negative impact of the hydraulics conditions not
allowed. Therefore reductions of the river flow profile should be compromised elsewhere (see design criteria, paragraph 2.4). Often it refers to locations which has been specified already to potential river widening projects like the other side of the river Meuse in Arcen (Berkhof et al. 2013). This screening criteria does not state the actual hydraulic impact of the river itself, but more the influence it has to future river widening projects. So, a design criteria is formed which state that the river flow profile should be equal or bigger with respect to the present situation, and a screening criteria is described which screens whether space will be used which was planned as a potential future river widening location.

An alternative will be assigned as good whenever it doesn’t influence the hydraulics at all. So, no extra location has to be considered. Bad can be assigned when a the total area is larger than a certain value. One has to keep in mind the three-dimensional effect of an alternative. Governing cross-sections are taken which do not represent the whole ring-levee section. Therefore the average cross-sectional area should be considered.

Limited screening time can be achieved by defining criteria such that it can be analysed with the data known beforehand. levee designs are drawn in governing cross-section during the available space screening step. By using this drawn area, and estimating the contribution of the effect in longitudinal direction, the data can be used. Furthermore, it is difficult to define a proper ratio between the alternative area with respect to the total river flow profile area. A much easier approach will be to constrict the river flow profile up to several meters from the present situation to the river-side. With this limitation, it is much easier to investigate the significance of this screening step. Hence, a decrease in area on the project side of the river (which will be compensated at the other side) will be investigated without considering the actual width of the river itself.

The embankment or front land along the present levees differs in width from just a few meters to several hundreds of meters. Often, cross-sections are taken without considering the whole embankment as it seems unrealistic that a levee alternative changes more than 50 meters to the river-side. Despite the latter opinion, there might be some extraordinary cases. But in most cases a good screening criteria will fit within 50 meters. Keeping in mind the different alternatives, a 20% area as maximum which will be doubtful. Less than 2% is assumed to be negligible. The criteria becomes:

<table>
<thead>
<tr>
<th>Percentage of cross-sectional area taken from the river flow profile between the present situation up to 50 meter to the river-side with the design water level is:</th>
</tr>
</thead>
<tbody>
<tr>
<td>+ Less than 2%;</td>
</tr>
<tr>
<td>0 between 2% and 20%;</td>
</tr>
<tr>
<td>– more than 20%.</td>
</tr>
</tbody>
</table>

### C.3.5. Pipes and cables

Basically, pipes and cables in, on top or nearby a levee are not wanted. A defect or renewing often entail an unwanted situation regarding the stability, height and/or the cover of a levee. Over the past few years many examples can be found of pipes breaking within a flood defences. One example is the crack in an old sewer pipe made of an asbestos material, 27th of December 2013. The sewer lays underneath the slope/tone of a green levee and leaked a lot of sewage water resulting in a decrease of the stability of the levee. After the break was detected, the water level of the river Meuse reaches one of its critical threshold for the Water board.

This was just one of the many examples the water board Peel and Maasvallei faces; other examples are the leaking sewage in Gennep, Defect pressure sewage near Heijen, one in Velden, Belfelt and another one in Neer, all damaged over the past few years (van Hal 2013).

In order to judge if and how a cable or pipe is allowed within the flood defence zone, they are divided into classes of priority (TAW 1994). Furthermore, the hydraulic guidelines (TAW 2003) defines different steps how to calculate and check the pipes and cables.
Besides the difference in priority, the assigned points should be related with respect to the importance in the whole screening process. Although many pipes and cables are unwanted, the total costs to replace them might be limited. Moreover, the assigned points of many pipes and/or cables should be less than the summation of all points per cable itself. The latter implication will be explained with an example: If a certain cable entails 10 points, than 4 cables of the same type should not be 40 points. Respectively, it is easier to replace 4 cables in total than one separate.

The priority list of cables and pipes from TAW (1994) is pointwise given below. Multiple cables and or pipes, as well as frequency over the levee section are evaluated later.

- Priority 1: All liquid- and gas pipes with a pressure of 10 bar or more and a diameter bigger than 0.3 meter;
- Priority 2: Parallel placed water-pipes or sewage system inside the outer slope or parallel levee;
- Priority 3: Parallel placed water-pipes or sewage system inside the crest, inner-slope or tone of the levee;
- Priority 4: A intersecting water-pipe or sewage system with the levee;
- Priority 5: Other water-pipes or sewage systems within the stability zone of the levee;
- Priority 6: Parallel placed gas-pipe in the outer slope or parallel levee;
- Priority 7: A intersecting gas-pipe with the levee;
- Priority 8: Parallel communication, electricity or glass-fiber cables in the outer slope or parallel levee;
- Priority 9: A intersecting communication, electricity, or glass-fiber cables with the levee;
- Priority 10: All other communication, electricity or glass-fiber cables within the stability zone of the levee;

There are two main types within the priority list: Parallel placed- and intersecting cables or pipes. The length of the pipes and cables parallel along the levee is an important aspect while the number of intersections is important for the priority types. The following assumptions are made concerning whether or not cables and pipes scores good, bad or doubtful.

- A maximum of one intersection per 100 meter levee of a priority of 4 is still good (hence, the priorities of 5 and more also);
- 4 or more intersections of a priority 4 pipe are assigned as bad;
- multiple cables and/or pipes within an intersection do not affect the points (the additional design c.q. construction aspects are limited);
- if a maximum of one pipe or cable parallel to the levee has to be replaced over its full levee reinforcement length, the assigned points are still good.
- if four or more pipes or cables parallel to the levee have to be replaced, is bad (they have to be in different pits)

Cables and pipes who lay inside the safety zone (C.10) of the new levee design should be replaced outside this zone (TAW 1994).

\[
Safety \text{ zone} = stability \text{ zone} + scour \text{ zone}
\]  
(C.10)

Here the stability zone is assigned as 4 times the levee height. The width of the scour-hole can be estimated by assuming a 45° angle form the bottom of the pipe/cable.

The total points to assign due to pipes and cables is expressed in equation C.11.

\[
Cables \text{ & Pipes} = \sum \frac{800}{priority \times L_{tot}} + \zeta \times \sum \frac{60}{priority}
\]  
(C.11)

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C.3.6. Trees
A tree nearby a levee can form a real threat to the flood defence safety. Especially overturning and uprooting of a tree can cause large gap formations. The size of such gap can be approximated by taking the width of the tree its crest and a depth of one meter according to the Hydraulic guidelines (TAW 2003). Secondly, a decrease in safety can be suspected regarding the permeability of the cover-layer caused by the presence of (dead) roots. Furthermore, trees might lower the quality of a grass cover and roots can harm structural elements.

![Figure C.6: Photo of a tree in Utrecht (the Netherlands) uprooted from a masonry historical qua-wall causing a huge gap during the storm conditions of 15 October 2013 (source: nu.nl)](image)

By following different steps, one is able to determine if a tree is allowed at a certain location depending on its type, health, position and the theoretical profile. Often structural measures have to be applied when a tree crosses the profile to create enough stability of the levee.

Unfortunately, these steps require a large amount of time. The design level to these steps do not coincide in this early stage of the screening procedure. However, there are many occasions of projects where trees have a significant influence.

There are different aspect used in where trees can be assigned as good, bad and doubtful. In order to make a distinction between larger and smaller trees, they are categorized in four different sizes. The distinguishing in size can be made via tree height, total tree width or by the trunk diameter. A Dutch contract-system (RAW) distinguish trees in sizes of 0-0,3 meter, 0,3-0,5 meter and larger than 0,5 meter. Hence, if such system might be used, it is beneficial to count trees with these trunk diameters. There is only one exceptions; The amount of rather small trees (smaller than 0,10 meter) have limited impact and relative easily replaceable. Therefore the following size distinguishing is made to the trunk diameter.

- < 0,10 Meter (negligible);
- 0,10 - 0,30 meter (small);
- 0,30 - 0,50 meter (medium);
- > 0,50 meter (large).

Next the number of trees still have to be assigned and linked to the size per tree. Many small trees can be even difficult as a few large one. Therefore several points are assigned per tree size. By counting

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6Recently an actual alternative is proposed in where 250 trees would remain, but costed around the 2 million Euro more (source: WPM).

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the number of total points and dividing them by the total length along the levee design, one is able to quantify whether or not the trees influence significant (scoring bad), influence insignificant (scoring good) or in-between.

The following assumptions are made when one tree scores good, bad or doubtful.

- Maximum of 5 small tree per 100 meter is assigned as good;
- Minimum of 20 small trees per 100 meter is assigned as bad;
- Maximum of 2 medium tree per 100 meter is assigned as good;
- Minimum of 8 medium trees per 100 meter is assigned as bad;
- Maximum of 1 big tree per 100 meter is assigned as good;
- Minimum of 4 big trees per 100 meter is assigned as bad.

By accounting trees with different sizes along the levee alternative, a certain total score can be obtained (see equation C.12).

$$
Trees = \frac{4 \sum_{i} trees(0.1 - 0.3)_i + 10 \sum_{m} trees(0.3 - 0.5)_m + 20 \sum_{n} trees(>0.5)_n \times 100}{length\ levee\ section} \quad (C.12)
$$

Where l, m and n are the number of trees along a ring-levee section respectively.

### C.3.7. Monuments

Some special attention is given to monuments which have often an extra requirements/difficulty. Generally, monuments like a century old city-wall, or an old peasant house, have large cultural value. They sometimes are sensitive for settlements or require additional permits.

Dealing with this criteria is very hard. One could state that a building which is an official (National or International) monument should be protected against a potential flood up to the required safety level. Another criteria could be thought of that it's not allowed to construct a flood defence against or within a certain range of the monument. However, across the river near Arcen a old castle is present which survived many floods in the pass. Nowadays, the castle is protected but gives a reduction to the river flow profile cross-section. Hence, the mentioned criteria seems logical, but there are cases in where one could consider differently.

So these criteria are formed with the consideration that there might be extraordinary cases. It is up to the designer to consider when these criteria should apply, and when not.

If more than 5% of the total length parallel to the river is a monument than:

- The alternative lays more than 20 meter from the monument.
- The alternative interface between the 5 and 20 meter.
- The alternative interface within a radius of 5 meter.

A value is assigned to the significance the monument(s) have in relation to the total ring-levee section. Tailor made solutions can be thought of with a relative small number of monuments. Hence, the alternative with a tailor made solution can still be favourable regarding their costs and should be analysed differently. The following criteria apply for smaller amount of monuments:

If less than 5% of the total length parallel to the river is a monument than:

- The alternative lays more than 5 meter from the monument.
- The alternative interface within a radius between the 0 and 5 meter.
- The alternative interface the monument directly.

The 5 and 20 meter are correlated to the ability to execute a certain alternative. Driving a sheet pile into the ground within 5 meters of a monuments is very risky and unwanted. Between 5 and 20 meters there it is still doubtful but possible and more than 20 meter is estimated to be a good distance to execute properly. From these two values, the constructibility include further assigned points.

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C.3.8. Nature
The importance of nature within a civil project has grown over the last few decades. There are various organisations who defines, maintain, facilitate and utilize these areas like the Natura2000 parks. Furthermore the government has designated a ecological structure in the Netherlands which connects natural area’s. A levee design alternative which goes through a natural area is less beneficial in contrast to an alternative who does not.

A distinction is made between areas who are internationally, nationally and regional recognized. The following points are assumed:

- 10 Points: Green levee type through a regional recognized natural area;
- 20 points: Structural levee type through a regional recognized natural area;
- 20 points: Green levee type through a national recognized natural area;
- 30 points: Structural levee type through a national recognized natural area;
- 30 points: Green levee type through an international natural area;
- 40 points: Structural levee type through an international natural area.

Assigned points related to the ecological structure:

- 20 points: Green levee crosses the ecological structure;
- 35 points: Structural levee crosses the ecological structure.

C.3.9. maintenance and inspection
There is a wide variation between levee design alternatives regarding the maintenance and inspection. Demountable and dynamic system have to be tested, some alternatives are hard to access during high water while other cost much more time to inspect and maintain properly. The following aspects are elaborated:

- Required inspection time;
- maintenance and inspection path width;
- closure during reactive or major maintenance;
- maintenance frequency;
- extensibility during major maintenance.

required inspection time
Many different types of levees with difficult structural elements resulting in a large time to inspect. A distinction has been made between four different levee types as described in appendix B. Points are assigned correlated with these types:

- 0 Points: Green alternative;
- 5 points: Structural permanent alternative;
- 10 points: Structural demountable alternative;
- 15 points: Structural dynamic alternative.

maintenance and inspection path width
The minimum required width of a inspection path is 3 meter. A larger path has additional advantages like applying a temporary system if failure occurs. The following points are assigned to the width of the maintenance and inspection path:

- 0 Points: Width > 4,50 meter;
- 5 points: 3,00 < Width < 4,50 meter;
- 50 points: Width < 3,00 meter;

If the maintenance and inspection path is not available during high water an additional 30 points have to be added. If the maintenance and inspection path is through someone’s else property, 20 points are assigned.

Closure during reactive or major maintenance
During reactive or major maintenance, there are occasions that a road or path have to be closed. For instance the installation of new rubbers for a glass wall system. Therefore the following points are given:
• 0 Points: no closure;
• 5 points: Bicycle/walking path closure;
• 10 points: road closure.

**maintenance frequency**

Levee design alternatives with a very short lifetime are unpleasant. Some systems are out of the marked which result in a different type of system. Depending on the suspected lifetime per alternative, the following points apply:

• 10 Points: Lifetime is within 20 years;
• 5 points: Lifetime is between 20 and 40 years;
• 0 points: Lifetime is more than 40 years.

**Extensibility during major maintenance**

The extensibility is an important aspect during the screening procedure. Levees who are hard to extend, like a dynamic system, enlarge the difficulty of maintenance in costs, execution time, hindrance, service costs, etc. Therefore additional points are assigned to include the extensibility within this screening criteria. However, it is very hard to state for all possible levee types their negative effects. Therefore, examples are given with their assigned points. The idea is, that the designer is able to assign points based on these examples.

• 0 Points: small measures to increase like:
  – Increase of green alternative with a green cover or maintenance path;
  – Increase of masonry wall;
  – extending existing demountable system (for instance new pillars and one extra beam with a stop-log system).

• 10 Points: Reasonable measures to increase like:
  – Increase of a concrete capping beam kind of structure;
  – increase of a green alternative with an bicycle- or traffic road on top;
  – change of a top element which can be replaces easily (e.g. a glass wall system).

• 20 Points: Difficult measures to increase like:
  – Piling of a sheet pile;
  – changing of a full dynamic system (e.g. Self closing flood barrier system).

**C.3.10. Basic costs estimation**

Besides the LCCA, costs have to be included within the screening procedure, too. If a levee design alternative does not score that great, but is relatively cheap, might still be realistic. A basic cost estimation can be realized by defining prices per unit to all kind of levee elements with a significant contribution. Multiplying unit prices to the quantity of the material within a running meter levee the total costs can be estimated. The unit costs should include all costs related to that specific elements such as: Purchase, planning, transport, executing, controlling, etc. Results of the basic cost estimate are shown in appendix G.

The alternatives will be screened based on the “cheapest” few levees. Defining the screening criteria based on only one alternatives may result in many doubtful or bad score. This is in contrast to the idea of having a variety between the alternatives. A percentages is assigned how large the difference is between the cheapest few and the other alternatives.

The levee design alternatives are assigned as “good” if they are cheaper than 120% of the average of the one third cheapest part of all alternatives. Between the 120% and 200% of this average becomes doubtful. Above the 200% are unrealistic.

**C.3.11. Connection between ring-levee sections**

If one levee reinforcement entails multiple sections, it is also important that they will be connected properly. A levee design alternative is realistic if it has the same levee alignment as an other realistic alternative. Although it seems logical, it is hard to frame it into a clear criteria. Having the previous criteria known, one is able to judge the particular section based on the scoring of the other sections.

For two sections at each side:

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• 15 Points per bad score of an other section;
• 5 points per doubtful score.

For one section:
• 20 Points per bad score of an other section;
• 10 points per doubtful score.

Per levee alignment option, all bad and doubtful scores can be added up and divided by the number of alternatives within this option.

Connection with the present situation:
• 15 Points: Intersection of a road;
• 5 points: Intersection of a bicycle/walking path;
• 10 points: Intersection of a lane of buildings;
• 5 points: Intersection of a park;
• 5 points: Distance is 10 meter or less;
• 10 points: Distance is between 10 and 40 meter;
• 15 points: Distance is more than 40 meter.
Hydraulic boundary conditions

Hydraulic boundary conditions are important aspect within the design process. Future extreme discharge conditions influence current statistics. Climate changes effect the discharge distribution to more extreme values and future river widening projects are planned along the river Meuse in the upcoming years. Moreover the current statistics have several errors.

There are large uncertainties in the forecasted water levels where the design will be based on. Nevertheless, only one design will be executed with one levee height per location.

This appendix report the deterministic and probabilistic formed hydraulic boundary conditions for both case studies Arcen and Venlo.

D.1. Deterministic hydraulic boundary conditions Arcen

Governing water levels of the river Meuse in Limburg change each year. Ongoing projects like Ooijen-Wanssum change the water level significantly. Furthermore there are changes in statistics and hydraulic models each year. Representative water levels coincide with a certain annual probability of exceedance connected to a specific discharge of the river Meuse. With an annual probability of exceedance of 1/1250 year, the river has an expected discharge of $4,000 \, m^3/s$ at this moment. This discharge data is estimated by extrapolating known high water levels. The water levels of 2013/2014 together with the discharges in Borgharen and the water level at Arcen are shown in table D.1. Ongoing project like Ooijen-Wanssum are not included in the water levels of 2013/2014.

It appears to be an approximately 2.8 meter water difference between the annual probability of exceedance of 1/2 and 1/1250 at Arcen. Unfortunately, water level differences deviate significantly along the river Meuse. Cities, villages and weirs forming a possible bottleneck in the river resulting in relative large water differences. In contrast, large floodplains diminish those deviations.

Due to climate changes an increase in discharge is assumed from $4,000 \, m^3/s$ at this moment to $4,200 \, m^3/s$ in the year 2050 and $4,600 \, m^3/s$ in the year 2100, all related to an annual probability of exceedance of 1/1250 year. The preferred estimated water levels are in the years 2020, 2070 and 2120. The best way to create the water levels is to model the whole river as detailed as possible with the well considered increase in discharge due to climate changes and river widening projects. Within this research governing water levels related to several years and failure probabilities are extrapolated with help of provided data in table D.1 and rules of thumb.

Today’s river management strategy is: “River widening measure whenever possible, increasing levees only when require”. It indicates the preferences of the Dutch government regarding levee reinforcements. River widening projects are the preferred measures to reducing flood risk above reinforcing

---

1\textsuperscript{A project just upstream of Arcen where one uses an old position of the Meuse as secondary channel during high water}
Table D.1: recurrence time with discharges at Borgharen-dorp and water level heights at Arcen (km. 120), source: WPM

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>1.627</td>
<td>14,80</td>
</tr>
<tr>
<td>5</td>
<td>1.987</td>
<td>15,59</td>
</tr>
<tr>
<td>10</td>
<td>2.260</td>
<td>16,10</td>
</tr>
<tr>
<td>20</td>
<td>2.533</td>
<td>16,52</td>
</tr>
<tr>
<td>30</td>
<td>2.685</td>
<td>16,71</td>
</tr>
<tr>
<td>40</td>
<td>2.786</td>
<td>16,83</td>
</tr>
<tr>
<td>50</td>
<td>2.865</td>
<td>16,93</td>
</tr>
<tr>
<td>75</td>
<td>3.008</td>
<td>17,05</td>
</tr>
<tr>
<td>100</td>
<td>3.109</td>
<td>17,14</td>
</tr>
<tr>
<td>250</td>
<td>3.431</td>
<td>17,37</td>
</tr>
<tr>
<td>1250</td>
<td>4.000</td>
<td>17,68</td>
</tr>
</tbody>
</table>

Levees. The strategy is accompanied by multiple governmental organisations like the province of Limburg as well as the Ministry of Infrastructure and the Environment of the Netherlands.

First the discharges related to the recurrence time are estimated by interpolating the discharges from table D.1. Figure D.1 shows the result of the interpolation and extrapolation via the Piecewise cubic hermite method (i.e. a spline where each piece is a third-order polynomial in hermite form). Limited data set, large uncertainties and the hydrodynamic behaviour of the river are present. At the moment only three values are used to extrapolate in which two of them are forecasted. Secondly, at a certain point, levees start to breach and/or overflow resulting in a decrease in discharge at the maximum top of a flood wave. Hence, although the “pchip”-method present the relation between years and forecasted discharge well, one should notice that these inter- and extrapolations entail a large error.
Interpolated discharges, related to time are used to form a factor as can be seen in table D.2. All years c.q. discharges coincide the same annual failure probability of 1/1250.

Table D.2: Interpolated and extrapolated discharges at Borgharen corresponding with an annual probability of failure of 1/1250 in the years 2020, 2070 and 2120 respectively

<table>
<thead>
<tr>
<th>failure probability</th>
<th>Time</th>
<th>Estimated discharges [m³/s]</th>
<th>factor [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/1250</td>
<td>2014</td>
<td>4.000</td>
<td>1,0000</td>
</tr>
<tr>
<td>1/1250</td>
<td>2020</td>
<td>4.023</td>
<td>1,0059</td>
</tr>
<tr>
<td>1/1250</td>
<td>2070</td>
<td>4.335</td>
<td>1,0837</td>
</tr>
<tr>
<td>1/1250</td>
<td>2120</td>
<td>4.788</td>
<td>1,1971</td>
</tr>
</tbody>
</table>

Over the years changes in discharge happen to all governing failure probabilities. Due to limited data a multiplication-factor (equation D.1) is calculated which can be used to find the correct discharge for a certain failure probability at a certain year. Although the accuracy is limited, it approaches the reality better than without this factor. Today's discharge is used as reference time ($t₀$). Table D.2 shows this factor.

\[
multiply \; factor = \frac{Q(t_i)}{Q(t₀)} \tag{D.1}
\]

Where:
- $Q(t_i)$ Estimated discharges at times $i$ [m³/s]
- $Q(t₀)$ Present discharge (4000) [m³/s]
- $i$ Years 2020, 2070 and 2120 respectively

Next, the multiplication factor is used to form discharges related to a specific failure probability and year. Figure D.2 shows the relation between discharges at Borgharen in the year 2014 and the failure probability (log-distributed). The pchip method fits the relation to interpolate/extrapolate the used data well.

Figure D.2: Interpolation of discharges at Borgharen with respect to failure probability in Arcen (km. 120) at Borgharen. The Piecewise cubic hermite interpolation method is applied.

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Discharges can now be found by using the relation with the failure probabilities and the multiplication factors. An overview of the required deterministic defined discharges is shown in table D.3.

Table D.3: recurrence time with discharges at Borgharen-dorp and water level heights at Arcen (km. 120), source: WPM

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1/250</td>
<td>3.431</td>
<td>3.451</td>
<td>3.718</td>
<td>4.107</td>
</tr>
<tr>
<td>1/500</td>
<td>3.667</td>
<td>3.689</td>
<td>3.974</td>
<td>4.390</td>
</tr>
<tr>
<td>1/1250</td>
<td>4.000</td>
<td>4.024</td>
<td>4.335</td>
<td>4.788</td>
</tr>
</tbody>
</table>

Before continuing the derivation of water levels, the finished steps are summarized:

- Data is collected for the present water levels (water levels Arcen (2014) related to discharges and the annual probability of failures);
- Forecast data is collected (future discharges related to a failure probability of 1/1250);
- Forecasted date is related to the time and interpolated to find the correct discharges with an annual failure probability of 1/1250;
- A multiplication factor is calculated;
- This factor is used to relate a governing discharges with failure probabilities at a certain point in time.

Next, formed discharges will be related to water levels at Arcen. Two methods have been used to interpolate/extrapolate the required data as can be seen in figure D.3. Both show an increase of the water level with respect to an increased discharge. However, the flood plane area increase with an increasing discharge. Flood planes starting to fill. Hence, it seems logical that there is a decrease

![Figure D.3: Extrapolation of the water levels in Arcen (km. 120) with respect to the discharges at Borgharen. Three methods are applied: Piecewise cubic hermite interpolation(pchip), splines and linear.](image-url)
D.2. Deterministic hydraulic boundary conditions Venlo

in the derivative of the relation. Therefore the linear inter-/extrapolation-method seems to be more appropriate than the spline-method.

Table D.4 shows the result of the deterministic determined water levels. This investigation included the water levels in the year 2120.

Table D.4: recurrence time with discharges at Borgharen-dorp and water level heights at Arcen (km. 120), source: WPM

<table>
<thead>
<tr>
<th>Failure probability</th>
<th>Water level 2013/2014</th>
<th>Water level 2020</th>
<th>Water level 2070</th>
<th>Water level 2120</th>
</tr>
</thead>
<tbody>
<tr>
<td>[-]</td>
<td>[m N.A.P.]</td>
<td>[m N.A.P.]</td>
<td>[m N.A.P.]</td>
<td>[m N.A.P.]</td>
</tr>
<tr>
<td>1/250</td>
<td>17.37</td>
<td>17.38</td>
<td>17.53</td>
<td>17.74</td>
</tr>
<tr>
<td>1/500</td>
<td>17.50</td>
<td>17.51</td>
<td>17.67</td>
<td>17.90</td>
</tr>
<tr>
<td>1/1250</td>
<td>17.68</td>
<td>17.70</td>
<td>17.87</td>
<td>18.12</td>
</tr>
</tbody>
</table>

Not all values are necessary for design purpose. With a base year of 2020, and a lifetime up to major maintenance till 2070 the governing water level is 17.53 meter N.A.P. (1/250). In 2070 the levee should fulfill requirements related to the governing water level of 18.12 meter N.A.P.

Rather large estimates are used when interpolating and extrapolating data. For instance, water levels relating to a failure probability of 1/250 have never been occurred yet. There might exist large flood plains which are in this stage not included within the Q-h relation. At the end the best way to investigate water levels for a levee design is by executing hydraulic models for the Meuse river.

D.2. Deterministic hydraulic boundary conditions Venlo

The case study Arcen is used during the development of the toolbox c.q. screening procedure and the optimization tool. Case study Venlo will be used as second case study. This paragraph contains the deterministic hydraulic boundary conditions for case study Venlo with the same manner as case study Arcen. The manner to find the hydraulic boundary conditions is reported in the previous paragraph D.1. Here results are presented only.

Table D.5: recurrence time with discharges at Borgharen-dorp and water level heights at Venlo (km. 120), source: WPM

<table>
<thead>
<tr>
<th>Recurrence time [year]</th>
<th>Discharge 2013/2014 $[m^3/s]$</th>
<th>Water levee 2013/2014 $[m \text{ N.A.P.}]$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>1.627</td>
<td>15.83</td>
</tr>
<tr>
<td>5</td>
<td>1.987</td>
<td>16.75</td>
</tr>
<tr>
<td>10</td>
<td>2.260</td>
<td>17.33</td>
</tr>
<tr>
<td>20</td>
<td>2.533</td>
<td>17.81</td>
</tr>
<tr>
<td>30</td>
<td>2.685</td>
<td>18.04</td>
</tr>
<tr>
<td>40</td>
<td>2.786</td>
<td>18.18</td>
</tr>
<tr>
<td>50</td>
<td>2.865</td>
<td>18.28</td>
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<td>75</td>
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<tr>
<td>100</td>
<td>3.109</td>
<td>18.52</td>
</tr>
<tr>
<td>250</td>
<td>3.431</td>
<td>18.77</td>
</tr>
<tr>
<td>1250</td>
<td>4.000</td>
<td>19.15</td>
</tr>
</tbody>
</table>

Table D.6 shows the result of the deterministic determined water levels for case study Venlo. This investigation includes the water levels in the year 2120.
Table D.6: recurrence time with discharges at Borgharen-dorp and water level heights at Venlo (km. 1080)

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>250</td>
<td>18,77</td>
<td>18,78</td>
<td>18,96</td>
<td>19,22</td>
</tr>
<tr>
<td>500</td>
<td>18,93</td>
<td>18,94</td>
<td>19,13</td>
<td>19,41</td>
</tr>
<tr>
<td>1250</td>
<td>19,15</td>
<td>19,17</td>
<td>19,37</td>
<td>19,68</td>
</tr>
</tbody>
</table>

D.3. Uncertainty in water levels

Uncertainties in hydraulic boundary conditions for a river basically depend on four different aspects:

- The uncertainty in statistics;
- the uncertainty in forecasted climate changes;
- the uncertainty in future river widening projects;
- the uncertainty in data due to future floods.

These uncertainties are considered separately but do have a certain dependency. If an expected increase in river discharge due to climate change is less than thought, it probably affect the political decisions whether or not a river widening project will be executed. Within this thesis, these parameters are assumed to be independent. The variation in the resulting distribution of the hydraulic boundary condition uncertainties will probably be overestimated. Although, the variation is always equal or larger than the uncertainties in statistics.

Combining uncertainties should be carried out carefully. All aspects require the same reference parameters before adding them up together. The chosen reference parameters are the deterministically determined increase of the discharge in line with the Dutch government. Equation D.2 shows the summation of all uncertainties including a deterministic increase ($\Delta h_{2,d}$). The deterministic scenario relates the well known Meuse discharge of 4,600 m$^3$/s with an annual probability of occurrence 1/1250 in the year 2100.

$$\Delta h_2 = \Delta h_{2,d} + \Delta h_{2,i} + \Delta h_{2,ii} + \Delta h_{2,iii} + \Delta h_{2,iv}$$  \hspace{1cm} (D.2)

Where:

- $\Delta h_{2,d}$: Deterministic reinforcement height [m]
- $\Delta h_{2,i}$: Reinforcement height difference due to uncertainties in river widening projects [m]
- $\Delta h_{2,ii}$: Reinforcement height difference due to uncertainties in statistics [m]
- $\Delta h_{2,iii}$: Reinforcement height difference due to uncertainties in climate changes [m]
- $\Delta h_{2,iv}$: Reinforcement height difference due to uncertainties in future floods [m]

D.3.1. Uncertainty in statistics

The river Meuse governing discharges are related to the recurrence times based on extrapolated hydraulic yearly data starting from 1911. An overview of the yearly maximum discharges at Borgharen over time is presented in figure D.4. The floods of 1926, 1993 and 1995 are clearly visible.

Different statistic distributions can be used which present these data. Jansen (2007) investigated uncertainties of the high water parameters during the process steps to calculate the design discharges. For instance the way of measuring over the years changed, indicating that these discharges do not represent the exact relative differences. Often applied distributions are the Gumbel distribution, Pearson III distribution, 3-par log-normal distribution, Weibull distribution and the exponential distribution. (de Wit 2004, Jansen 2007). Table D.7 shows an overview of different kind of distributions, their average discharge and 95% confidential interval.

Besides the different kind of distributions there are also various researches carried out to forecast the governing discharges of the river Meuse (table D.8). The 95% confidential interval of the hydraulic boundary conditions (Hydraulische Randvoorwaarden 2001 or HR2001) match the values of table D.7.

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D.3. Uncertainty in water levels

Moreover the average is used to form "best" set of forecasted discharges. Hence, uncertainties of the river discharge is caused partly by the differences in statistic distributions.

Another important aspect is the use of relative short time period of data. The latter seems logical as a recurrence time of 1250 years is wanted when approximately 100 years of data is available (de Wit 2004).

The actual distribution which fits exactly to the data is not known and can't be known (i.e. after each

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significant flood, the data and hence the distribution differs). The distributions which are in the neighbour-
hood (e.g. Gumbel, Pierson III, 3-par log-normal) have some errors. Often an error is simulated
with a Gaussian, or normal distribution. Therefore a normal distribution is assumed which represent
the error due to the differences between the distributions. The governing discharge of the HR2006
water level is used corresponding with the 1/1250 flood (see table D.9).

Table D.9: Uncertainty in forecasted discharge of the HR2006 water levels with a 1/1250 flood. Water levels are specified for
the case study Arcen.

<table>
<thead>
<tr>
<th>Q</th>
<th>h</th>
<th>Δh</th>
</tr>
</thead>
<tbody>
<tr>
<td>[m³/s]</td>
<td>[m N.A.P.]</td>
<td>[m]</td>
</tr>
<tr>
<td>low 95% interval</td>
<td>3.250</td>
<td>17,24</td>
</tr>
<tr>
<td>high 95% interval</td>
<td>4.705</td>
<td>18,07</td>
</tr>
</tbody>
</table>

With equation D.3 and the stated values the standard deviation can be found. With a Z-value of 2.81
the standard deviation can be found of 0.15 meter. As we would like to have the uncertainty of the
forecasted discharge only, the mean value of the normal distribution can be set to zero.

\[
Z = \frac{X - \mu}{\sigma_x} \sim N(0,1)
\]  

(D.3)

Where:
- Z Value to look up in the standard normal distribution-table
- X value of a confidential interval
- \( \mu_x \) Mean value
- \( \sigma_x \) Standard deviation
- N(0,1) Standard normal distribution (\( \mu = 0, \sigma = 1 \))

Till now, only the present day discharges are discussed with their uncertainties. Additionally, climate
changes and other uncertainties are included as well into the design of a levee. It even appears to be
the case that differences between various climate scenarios are larger with respect to the uncertainties
discussed above. (de Wit 2004).

**D.3.2. Uncertainty in forecasted climate changes**
The river Meuse has a seasonal discharge regimes with low-flows during summer and high-flows during
winter. It is most likely that differences in seasonality enduring due to temperature increases and
changes in the rainfall-evaporation regimes (de Wit et al. 2007). Based on forecasts of climate pre-
diction models, changes in temperature, precipitation and evapotranspiration. These climate changes
effecting the extreme discharges of the river Meuse (van Pelt et al. 2009). The primary changes are
related to the variance of the discharge (or precipitation/evaporation) rather than an increase in aver-
age. Currently the Dutch government applies a so called long term precautionary principle regarding
the river discharges. The discharge in this case is estimated to be on the conservative side instead of
using the average. The Dutch government currently uses the 2006 scenario’s of the Royal Netherlands
Meteorological Institute or KNMI² (G, G+, W, W+) which are indicated together with data starting from
1911 in figure D.5. Part of a summery of the KNMI scenarios are gathered in table D.10.

**Relation between temperature increase and river Meuse discharge**
There are various rules of thumb to estimate the increase in discharge related to the temperature
and/or to the precipitation differences in the upcoming period. de Wit et al. (2006) defined on average
precipitation increase in relation to the relative temperature increase of 8%°C⁻¹. The percentage in-
crease of the yearly maximum 10-days winter precipitation of the river Meuse basin can be estimated
as the same percentage increase of the extreme discharges of the river Meuse (de Wit et al. 2007).
The latter rule of thumb is partly based on studies of CHR (Grabs et al. 1997) which connected the
discharge and temperature changes directly. Here a temperature increase of 1°C result in an extreme

²KNMI: Koninklijk Nederlands Meteorologisch Instituut

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D.3. Uncertainty in water levels

Figure D.5: Time series of observed precipitation [mm per 3 months] and temperature with the KNMI 2006 scenario’s in coloured dashed lines. The gray areas present the interannual variability (van den Hurk et al. 2006)

Table D.10: Summery of climate change variables of the KNMI 2006 scenario's (de Wit et al. 2006)

<table>
<thead>
<tr>
<th>Scenario 2050</th>
<th>G</th>
<th>G+</th>
<th>W</th>
<th>W+</th>
</tr>
</thead>
<tbody>
<tr>
<td>Winter average temperature</td>
<td>+0.9°C</td>
<td>+1.1°C</td>
<td>+1.8°C</td>
<td>+2.3°C</td>
</tr>
<tr>
<td>Winter average precipitation</td>
<td>+4%</td>
<td>+7%</td>
<td>+7%</td>
<td>+14%</td>
</tr>
<tr>
<td>Winter 10-days precipitation exceeding 1 every 10 years</td>
<td>+4%</td>
<td>+6%</td>
<td>+8%</td>
<td>+12%</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Scenario 2100</th>
<th>G</th>
<th>G+</th>
<th>W</th>
<th>W+</th>
</tr>
</thead>
<tbody>
<tr>
<td>Winter average temperature</td>
<td>+1.8°C</td>
<td>+2.3°C</td>
<td>+3.6°C</td>
<td>+4.6°C</td>
</tr>
<tr>
<td>Winter average precipitation</td>
<td>+7%</td>
<td>+14%</td>
<td>+14%</td>
<td>+28%</td>
</tr>
<tr>
<td>Winter 10-days precipitation exceeding 1 every 10 years</td>
<td>+8%</td>
<td>+12%</td>
<td>+16%</td>
<td>+24%</td>
</tr>
</tbody>
</table>

discharge increase between the 5 to 8% and a 10% discharge increase with 2 degrees. When uncertainties in temperature (or precipitation) due to climate changes are known, these rules of thumbs provide an easy first estimate. Many scenario’s are assigned and/or restudied to future climate changes over the years often in cooperation with the KNMI or the IPCC³.

Hence, the increase in discharge over the years due to climate changes can be calculated with rules of thumb knowing the temperature and/or precipitation increases. Next the uncertainties due to these climate changes are described. Combining the relation between temperature/precipitation with the discharge and the uncertainties in climate changes an actual distribution with parameters can be chosen depending on the time c.q. probability of a flood.

Uncertainties of climate changes

In order to investigate the uncertainty relation between climate changes (i.e. temperature-, precipitation- and evapotranspiration changes) and impact of the water levels near the case studies various model can be used. According to Booij (2004) and van Pelt et al. (2009) the HBV⁴-, rainfall run-off model of the SMHI⁵ seems to be appropriate and has been applied to the river Meuse basin. Briefly, the model simulates river discharge using precipitation, temperature and evapotranspiration data as input. Especially the study of Booij (2004) gives some insight into the uncertainties of extreme river discharges related to climate changes. The following is cited by Booij regarding these uncertainties:

"The uncertainty in the estimation of the 100-year return value is 20% (one-sided) using a 90% confidence interval and employing a 30-year series. The uncertainty is 25% when using a 95% confidence interval."

These uncertainties can only be reduced by applying longer time series than the current used 30 years of data. Furthermore errors due to extrapolation and precipitation measures seems to be of significant

³IPCC: Intergovernmental Panel on Climate Change
⁴HBV: Hydrologiska Byråns Vattenbalansavdelning model
⁵SMHI: Swedish Meteorological and Hydrological Institute

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importance, too. Measuring the precipitation (probably also the evapotranspiration but isn’t discussed specifically in the article) enhance often a local quantity rather than an actual average quantity. This lead to an overestimation of the variance and is more significant as the model errors⁶.

**Relation between climate changes uncertainties and river discharge**

The uncertainties due to climate changes are now known for a 100-year return period extreme discharge. Before continuing to appoint the appropriate distribution and parameters, the described rules of thumb are used to quantify the increase in extreme discharges of the river Meuse related to time. Unfortunately, there is a big difference in the way results are presented. The temperature increase (van den Hurk et al. 2006) is related to scenario’s in the year 2050 and 2100, while the results of the uncertainties are related to a 100-year return period (Booij 2004).

To deal with this inconsistency between data, first the increase in discharge via rules of thumb are used with a 1/100 flood. Here the expected value of the discharge increase is calculated by averaging all KNMI scenario’s. Secondly, the uncertainties are included depending on the temperature/precipitation increase. Hence, the variance will increase over time. When establishing a function depending on time, one is able to use the results without linking the uncertainties in climate changes with the probability of occurrence of a flood. At the end the uncertainties will be linked to the water level heights specified for Arcen.

The 2014 discharge of a 100-year return period is 3.109 m³/s (table D.1). Data obtained from the KNMI scenario’s are calibrated from the year 2006 while this discharge represent the 100-year flood in 2014. The average 10 days precipitation increases about 7,5% once every 10 years in the year 2050 and 15% in the year 2100 (all four scenario’s are assumed to be equal in the average discharge: G, G+, W and W+). Figure D.6 shows the average, the maximum scenario (W+) and the minimum scenario (G) graphically. A 90% confidence interval is drawn on the average (see blue dotted lines). According to Booij (2004) the interval has a spread of 20%. In this case, the 20% difference is taken from the discharge increase over time. An increase of 20% above the maximum scenario and below the minimum scenario are shown as well.

![Figure D.6: Forecasted increase of the river Meuse discharge (average, maximum KNMI scenario and minimum KNMI scenario) over time due to climate changes including the 20% intervals of both the average, as the total KNMI scenario’s](image)

Unfortunately, there is still a large difference between the scenario’s obtained from the KNMI and the 90% interval of the climate change uncertainty. It is questionable how much the researches are connected to each other. For instance, which scenario’s have been used during the rainfall run-off model, and what is the uncertainty of the rules of thumb gathered? At this point a choice have to be made to which kind of distribution and related parameters suits best the uncertainties in forecasted climate changes. The 20% of the average seems an underestimation of the total variability while the 20% uncertainty of (van Pelt et al. 2009) describe a model improvement but haven’t assigned a new uncertainty analysis regarding the relation of climate changes and extreme discharges

⁶Note that the research of (van Pelt et al. 2009) describe a model improvement but haven’t assigned a new uncertainty analysis regarding the relation of climate changes and extreme discharges
above and below the minimum and maximum discharges seems to be an overestimation. Assumptions have to be made how big the changes are to have a bigger and lower discharge with respect to the maximum and minimum scenario's.

The chance that future river Meuse discharge is higher as the discharge related to the maximum KNMI scenario is assumed to be 2,5%. The same chance holds for the chance of a lower discharge than the minimum scenario.

Secondly a distribution has to be chosen. Again, at this point, with the data and researches known, it is very hard to argue which distribution fits best.

A uniform distribution is assumed with the intervals given in the previous framed box.

The distribution as well as its parameters should not be used in further investigations due to the assumptions made. If there is a large influence of the climate changes (as well as the other uncertainties) regarding the discussion making process between levee design alternatives, a recommendation can be defined to investigate the distribution and its parameters more thoroughly.

Relation between discharge distribution and water levels

The increase in discharge due to climate changes are estimated to be uniformly distributed with a 95% confidence interval between the most outer KNMI scenario's of 2006 (i.e. 2,5% of the upper bound and 2,5% of the lower bound). The yearly increase in discharge is used to find the corresponding discharges related to a 1/1250 flood in the year 2120 (the governing design discharge for major maintenance). The year 2100 is also given to show the relation with the 4.600 m³/s discharge. The increase in temperature is linear, leading to a linear relation regarding the increase in precipitation. This lead to a linear increase in discharge. Hence, the increase in discharge over the flood probability is assumed to be linear. However, climate changes may cause a different precipitation distribution. The increase in precipitation due to climate changes may be more in some areas than others. It means that the linearity is more an assumption rather than a scientific based theory. Table D.11 shows the results.

<table>
<thead>
<tr>
<th></th>
<th>$\Delta Q$ 95% interval</th>
<th>Q(2100)</th>
<th>Q(2120)</th>
<th>h(2120)</th>
<th>$\Delta h$(2120)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>[m³/s/yeár]</td>
<td>[m³/s]</td>
<td>[m³/s]</td>
<td>[mN.A.P]</td>
<td>[m]</td>
</tr>
<tr>
<td>Average</td>
<td>4,86</td>
<td>4,515</td>
<td>4,612</td>
<td>18,02</td>
<td>-0,10</td>
</tr>
<tr>
<td>Standard</td>
<td>4,600</td>
<td>4,788</td>
<td>18,12</td>
<td>0,00</td>
<td></td>
</tr>
<tr>
<td>minimum</td>
<td>2,37</td>
<td>4,235</td>
<td>4,279</td>
<td>17,84</td>
<td>-0,28</td>
</tr>
<tr>
<td>maximum</td>
<td>8,07</td>
<td>4,871</td>
<td>5,036</td>
<td>18,25</td>
<td>+0,13</td>
</tr>
</tbody>
</table>

D.3.3. Uncertainty in future river widening projects

The Dutch government investigated area of the river Meuse in Limburg to river widening possibilities via the Delta-program. The delta committee debate with people from the Ministry of Infrastructure and the Environment, Province, municipalities and water boards to define whether an area has potentials for future measures affecting the high discharges positively. By including regional partners during the development process of preferred strategies one creates a large possible regional support. The preferred strategy is chosen and all kind of measures to the river embankment as well as the levees are assessed. The preferred strategy’s state: “River widening measures when possible, reinforcing levees when needed” (DPR 2013). In 2015, the cabinet has to approve the defined strategies by the delta

7 Dutch: Voorkeursstrategie or VKS
8 Dutch: verruiming waar het kan, dijken waar het moet.
committee. A final balance will be made between levee reinforcing, river widening measures and/or a mix of both during future development of the strategy on smaller scales.

The areas along the river Meuse are assigned as green, yellow and red, categorized to indicate whether or not it is social accepted to adapt a river widening project within this area. How to interpret these colors is described below. Maps have been made to illustrate the areas like the one around Arcen (see figure D.7).

- **Green**: Based on today’s insights and information these areas can be assigned as social accepted to perform river widening measures into further development of the strategy (which doesn’t imply that they already have a green light for execution);
- **Orange**: Based on today’s insights and information these areas are questionable by one or more participants to classify it as green or red area⁹;
- **Red**: Based on today’s insights and information these areas do not have any social support by one or more participants.

Furthermore there are several ongoing projects influencing the river Meuse hydraulics. These projects belong to the so called “reference-plus” situation of the river Meuse and are actualized to the second phase up to the year 2020. Some projects are already implemented while others will be executed in a relative short period. The reference-situation is illustrated in figure D.7 with **yellow**. Most important projects are:

- Ooijen-Wanssum;

⁹Berkhof et al. (2013) described this category as yellow but is inconsistent with the maps which indicate this category as orange. At the moment their working to upgrade the document in such a way that there will be no inconsistencies any more.

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D.3. Uncertainty in water levels

- Lus van Linne (Roermond);
- Nevengeul Stadsweide (Roermond);
- Maaspark Well;
- Hoogwatergeul Raaijweide;
- Several Flemish projects along the Meuse.

Future river widening projects may influence the governing water levels significantly resulting in different major maintenance costs. The delta-committee reported the calculated water levels per scenario due to river widening projects. Scenarios are related to the mapped coloured areas: Green, orange and red. Figures D.8, D.9 and D.10 showing the hydraulic impact due to river widening project scenarios in the years 2030, 2050 and 2100 respectively. Both case studies Arcen and Venlo are present within these figures.

The graphics indicate the water levels of the river Meuse. The horizontal axis indicate the longitudinal direction of the river Meuse in kilometres and the vertical axis shows the water levels in meters from N.A.P. The lines represent different water levels related to possible future widening projects c.q. a difference in safety levels. Each is described below:

- **Red triangular**: Indicate the water height which can be retained by the present levee situation\(^{10}\);
- **black dotted line**: Design water levels according to SSK (Sluitstukkaden);
- **blue line**: Water levels in accordance with the reference situation including climate changes;
- **green line**: Water levels in accordance with the reference-plus situation including the climate changes and the yellow river widening projects up to 2020;
- **black bolt line**: Water levels in accordance with the preferred strategy including the yellow, green, orange and red areas;
- **Orange bolt line**: Water levels in accordance with the preferred strategy including the yellow, green, orange and red areas but with a new analysis norm (new safety approach) with an annual probability of a flood of 1/500 years and 1/1250 years along cities (like Arcen and Venlo).

\(^{10}\)Please note that these heights are formed with the assumption that all levees fulfil the present safety requirement of 1/250. So, the actual levee design alternatives formed for the case study Arcen should have around the same height as these triangular. One can see a water level around the 17,50 meter N.A.P. at km. 120 (Arcen) which is in line with the extrapolated value in paragraph D.1.

Figure D.8: Effect of possible future river widening project regarding the water levels in the year 2030

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These lines indicate the minimum and maximum hydraulic effects by possible river widening projects over the upcoming decades. Of course there are some uncertainties of the hydraulic models which have been used but for now it gives a clear view.

Figure D.9: Effect of possible future river widening project regarding the water levels in the year 2050

Figure D.10: Effect of possible future river widening project regarding the water levels in the year 2100

The minimum and maximum water level differences are estimated for all ring-levees along the Meuse in Limburg (and partly the province of Noord Brabant) with the reference-plus and the preferred strategy situation. The water level differences ($\Delta h$) of the ring-levees of Arcen and Venlo are summarized in table D.12. These numbers are highly indicative and will be updated after model analysis which will be completed in 2014 (Berkhof et al. 2013).

Unfortunately there are no values given to water level differences when only the “green” or “green + orange” areas are used for river widening projects. This is probably related to the earlier defined slogan: “River widening measures when possible, reinforcing levees when needed”. Hence, analysing only part of the areas is contradictory to this statement and therefore not included in the documents (yet).

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Table D.12: Prognosis of the minimum and maximum water level differences at Arcen and Venlo related to a certain river widening situation (Berkhof et al. 2013)

<table>
<thead>
<tr>
<th>Ring-levee</th>
<th>Situations</th>
<th>Δh at year:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>2030</td>
</tr>
<tr>
<td>Arcen 65</td>
<td>Reference-plus</td>
<td>0.17</td>
</tr>
<tr>
<td></td>
<td>Preferred strategy</td>
<td>0.07</td>
</tr>
<tr>
<td>Venlo-Velden 68</td>
<td>Reference-plus</td>
<td>0.47</td>
</tr>
<tr>
<td></td>
<td>Preferred strategy</td>
<td>0.22</td>
</tr>
</tbody>
</table>

Having a basic understanding of the ongoing developments regarding the river widening projects, assumptions have to be made to find a related distribution and parameters. It is likely that future projects affect the water level difference between the reference-plus and the preferred strategy situations. However, it is uncertain how affective the ongoing projects (included in the reference-plus situation) actually are. It will not be the first time that a certain prognosis to a lower water level ended up to be much larger as the actual one. At the other hand one could argue that today’s computer analysis are much more sophisticated reducing this error significantly. Still there is a change that these water level changes due to ongoing river widening projects are too optimistic. Hence, a certain confidential interval is assumed to include these uncertainties within the total distribution. The following assumption related to the river widening uncertainty can be made:

There is a 5% chance that at the end of planning, developing and executing the river widening projects till 2100, the water level difference is larger than the assigned reference-plus situations.

The same reasoning can be considered at the minimum water level difference which is also a forecasted set of numbers. Having in mind the probable difficulty to achieve river widening projects at the red areas, the probability to come underneath these minimum values can be estimated as smaller. Therefore the following assumption is applied:

There is a 5% chance that at the end of planning, developing and executing the river widening projects till 2100, the water level difference is smaller than the assigned preferred strategy situations.

The river widening projects are described and calculated up to the year 2100. However, during the case study, the analysis period end in the year 2120. The difference in water level between a 4.600 and a 4.788 m³/s discharge (forecasted discharges in the years 2100 and 2120 related to an annual failure probability of 1/1250) is about 0.05 meters if the Q-h relation is used from figure D.3. Ongoing river widening projects, like Ooijen-Wanssum, should change the Q-h relation positively. Larger areas will be flooded which decreases the Q-h slope. The 0.05 meters of increase therefore can be seen as a "worst case" scenario.

Next, the type of distribution has to be assumed, which probable is one of the most subtle parts of the estimation of uncertainties in hydraulic boundary conditions. Besides the lack of decent data (only two strategies), it is hard to describe politics. At last there should be a certain dependency between the future actual discharges and future actual river widening projects (i.e. larger discharge increases lead to probably bigger chance of river widening projects). Again a worst case can be thought if a uniform distribution is chosen. When using common scense, one can state that the probability the actual increase in water height is larger between the upper and lower bound than at these boundaries itself. Secondly, there is a relative big chance that the green areas will be used as river widening locations (Unfortunately, there is no strategy which shows the water levels with only the green areas). To cope with a bigger chance in the middle (with the maximum slightly to the lower boundary), a triangular distribution is assumed. The peak is assumed to be 2/5 of the total difference between the outer

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parameters of the distribution. This reasoning lead to the parameters stated in table D.13.

The current prospected water level increase (without any river widening project) is extracted from the parameter values such that the zero reflects the situation related to a 4.600 m³/s discharge in the year 2100.

Table D.13: Triangular distribution parameters to translate the expectation of future river widening projects along the river Meuse in Limburg specified for the Arcen case study.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Water level increase</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>0,39-0,64=-0,25</td>
<td>[m]</td>
</tr>
<tr>
<td>b</td>
<td>0,74-0,64=0,10</td>
<td>[m]</td>
</tr>
<tr>
<td>c</td>
<td>0,53-0,64=-0,11</td>
<td>[m]</td>
</tr>
</tbody>
</table>

D.3.4. Uncertainty in data due to future floods
Sudden changes in normative river discharge (e.g. high waters of the river Meuse in the years 1993, 1995, 2002 and 2003) alternate the probability a discharge occurs (de Wit 2004). A future large flood might change extrapolation of present data significantly. Visa versa, if no extreme floods occur, the probability of a certain flood decreases. Hence, besides the uncertainties of the present data, also future data create a certain spread. Unfortunately however, it is strange to assume future floods realistically. Whenever a flood occur, which changes the statistics significant, the probability that flood occurs change itself.

Of course there is the ability to investigate extreme value distributions by altering known discharge data. Leander and Buisman (2008) ran multiple simulations in where they investigated the sensitivity of extreme winter maxima of 10-day precipitations based on historical data. They concluded the following:

"The influence on the composition of the typical 1250-year event as well as the entire Gumbel plot was assessed. It was found that the effect of the altered algorithm on the 1250-year event was within 5%. For short and moderate return periods no effect on the 10-day maxima was found."

Unfortunately, there are some aspects which limit the ability to use this conclusion into the hydraulic boundary condition uncertainties. Leander and Buisman (2008) emphasised their investigation to the basin-average precipitation together with historical data in the period 1930-1998. Secondly, they only described the Gumbel distribution which actually is one of the possible distributions to chose. Mostly, it is unclear which part of there findings is already taken into account via the uncertainties in forecasted discharge.

A research to actual uncertainty due to future possible floods, by altering historical data, have to be carried out including the dependency of different distribution types. As the additional uncertainties due to future possible flood probably is relatively small regarding the other uncertainties, and the fact that their contribution within the uncertainties in forecasted discharge is possible, they are not taken into account within this research.

D.3.5. Results Monte Carlo simulations
The uncertainties in hydraulic boundary conditions mentioned above are simulated with a Monte Carlo-model (see paragraph D.4.1). Figure D.11 shows the results of this simulation. The relatively large spread of the expected levee reinforcement in 2070 is clearly visible. The large spread in combination with the realistic possibility of a relative large levee reinforcement increase create extra awareness to the levee design alternative analysis. Some alternatives which are limited in possible extension (e.g. dynamic levees), or have large investments during major maintenance, might be more doubtful. Of course there are many limitations/assumptions present with these results. Nevertheless, it gives a reasonable idea to the variety of the uncertainties regarding hydraulic boundary conditions.

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D.3.6. Results FORM simulations

A sensitivity analysis has been carried out with a FORM-simulation. The influence coefficients are presented in table D.14. It seems that all uncertainties contribute significantly. The impact of the future river widening projects is limited due to the triangular shape of the distribution.

Table D.14: Sensitivity analysis of hydraulic boundary condition uncertainties

<table>
<thead>
<tr>
<th>Uncertainty</th>
<th>Influence factor</th>
<th>Importance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Forecasted discharges</td>
<td>0.5131</td>
<td>31%</td>
</tr>
<tr>
<td>Future climate changes</td>
<td>0.7774</td>
<td>47%</td>
</tr>
<tr>
<td>Future river widening projects</td>
<td>0.3638</td>
<td>22%</td>
</tr>
</tbody>
</table>

D.3.7. Limitations of the uncertainty analysis

Unfortunately the done simulations still holds quite large assumptions and possible errors. The most important ones are summarized below:

- The river widening projects distribution is assumed to be triangular distributed; with an estimated mode;
- the data provided for the river widening projects were very limited both in quality as quantity;
- future climate changes are assumed to be uniformly distributed. An other type of distribution probably result in a more narrow result;
- KNMI scenario’s and rules of thumb are used to estimate the required probabilistic data. A research to climate change effect to extreme discharges with a rain run-off model probably result in more accurate results;
- only around 100 years of data is used to extrapolate/interpolate the discharges c.q. water levels. More data (and a better understanding of the influences between the distributions used) create a more accurate outcome;
- the governing discharges from uncertainty analyses are directly translated to a water level at a certain location. However, the Q-h relation changes due to river widening projects which are executed right now, as well as in the future;
- possible changes in the current might results in differences in water levels which are not taken into account;

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• all uncertainties are assumed to be independent. However, there is probably a dependency between climate changes and future river widening projects;
• river widening projects in other countries are not considered;
• there are some minor errors present due to the use of different resources (e.g. a distribution of the probability of occurrence is formed with waterbase data related to discharges in Borgharen, while the water board provided discharge data correlated to the actual locations of the case studies).

D.4. Calculation methods
This paragraph describes shortly the probabilistic simulation methods; both the Monte Carlo (Level III) and the FORM (Level II) simulations. Additionally most important distributions are introduced.

D.4.1. Monte Carlo simulation
A Monte Carlo simulation is a mathematical technique to assess a risk analysis. It is a widely used method in all kind of risk assessment fields and used when problems are too complicated to solve analytically. The name refers to a city in Monaco famous of its casinos. Basically, the method uses the possibility to draw a random number of a uniform probability density function between 0 and 1.

An arbitrary probability distribution function \( F(X) \) is always distributed uniformly between zero and one:

\[
F_X(X) = X_u
\]  
(D.4)

By drawing randomly numbers of an uniform probability density function, one is able to generate the variable \( X \), regardess the type of the distribution function \( F(X) \). Hence, the variable \( X \) can be achieved by taking the inverse of the distribution function of the uniformly distributed variable (CUR 2006):

\[
X = F_X^{-1}(X_u)
\]  
(D.5)

The distributions are summarized in paragraph D.4.3. The open earth tool\(^{11}\) model tool from Deltares is used to perform the Monte Carlo simulation. The adjusted matlab scripts are shown in paragraph D.6.

The number of simulations (\( n \)) is very important in the Monte Carlo method as too little results in significant errors. The appropriate number of simulations depends on the desired accuracy described with a confidence interval. With help of the central limit theorem, one knows that the sum of a large number of independent stochastic variables, which have a variance smaller than infinity, are approximately Gaussian distributed (with mean 0 and variance \( \sigma^2/n \)). A confidential interval is assumed:

\[
P(-2.575 \leq G \leq 2.575) \approx 0.99
\]  
(D.6)

The error \( (\epsilon_n) \) of the outcome of the Monte Carlo simulation with a probability closed to 99% can be approximated with (Lapeyre 2007):

\[
|\epsilon_n| \leq 2.575 \frac{\sigma}{\sqrt{n}}
\]  
(D.7)

Assuming a large standard variation of, lets say 0,60 meter, and a maximum error of 0,02 meter, the minimum number of simulations becomes: 5.968 \( \approx \) 6.000. The 0,60 meter is estimated as an upper bound value, so if one checks later, the minimum required simulations is almost always guaranteed.

After the simulation a variance have been found of 0,0430 for case study Arcen and 0,0478 for case study Venlo. Both result in a smaller standard deviation as previous assumed:

\[
\sigma_{HBC, Arcen} = \sqrt{0.0430} = 0.21 < 0.60
\]  
(D.8)

\[
\sigma_{HBC, Venlo} = \sqrt{0.0478} = 0.22 < 0.60
\]  
(D.9)

\(^{11}\)A free online tool used via Matlab software, created by Deltares.

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D.4. Calculation methods

D.4.2. FORM simulation

The “Official” aim of a FORM analysis, or level II method analysis is to find the value of the reliability index ($\beta$) resulting in a probability of failure.

$$\beta = \frac{\mu_Z}{\sigma_Z}$$  \hspace{1cm} (D.10)

The method estimates the reliability index by searching the minimum distance from the origin to the failure space. Finding the reliability index for a linear limit state function with normally distributed stochastic variables is straightforward. In this case, the limit state function itself will also be normally distributed. By transforming the limit state function to the failure space, or U-space (D.11) the normal distributed limit state function is linear.

$$U_i = \frac{X_i - \mu_i}{\sigma_X}$$  \hspace{1cm} (D.11)

However, often there is a dependency between parameters or different types of distributions. In these cases a non-linear limit state function occurs in the failure space. To estimate the reliability index, it is necessary to make a linearisation of the limit state function. Further information and equations regarding the linearisation during the FORM analysis can be read in CUR (2006). In order to create enough accuracy, iterations have to be performed with help of a design point ($X_i$). This point has the greatest joint probability density in the failure space. The coordinates of the design point ($u_1, u_2$) are:

$$(u_1, u_2) = \left( -\frac{\sigma_R}{\sigma_Z} \beta, \frac{\sigma_S}{\sigma_Z} \beta \right) = (\alpha_1 \beta, \alpha_2 \beta)$$  \hspace{1cm} (D.12)

in which the $\alpha_i$ is the influence coefficient. During each iteration step the design point differs as well as the influence factors. If the difference becomes small an approximation of the design point has been found. Some variables in the limit state function having more influence than others. The influence factor shows the relative dependency between the variables.

The FORM method will be used as sensitivity analysis despite it’s officially aim to estimate the probability of failure. Each variable within the limit state function will be provided with an influence factor during the iteration steps. The FORM method to execute a sensitivity analysis is chosen as it is included to the OpenEarth tool of Deltares. Unfortunately, the method, derives the sensitivity of a stochastic variable with respect to the failure of the limit state function. This is assumed to be acceptable with the scale of accuracy the uncertainties are formed in this research.

D.4.3. Distributions

Within this appendix, and during the simulations, various distribution types are stated. Present day statistics involve a large amount of distributions. Here, the most important distributions are introduced briefly. In this case the cumulative distribution function ($F(X)$) of the described ones are given. When taking the derivative of these functions, the probability density function ($f(X)$) can be found.

**Uniform distribution**

A uniform distribution describes an event to have an equal probability between the parameters $a$ and $b$. The cumulative distribution function can be described like:

$$F(X|a, b) = \begin{cases} 
0 & \text{if } x < a \\
\frac{x-a}{b-a} & \text{if } a \leq x < b \\
1 & \text{if } x \geq b
\end{cases}$$  \hspace{1cm} (D.13)

Where:

- $x$ Random variable
- $a$ lower boundary
- $b$ upper boundary

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**Triangular distribution**

The triangular distribution, as the name says so, is triangular shaped as a probability density function. It has a lower boundary \((a)\), an upper boundary \((b)\) and a mode \((c)\) which does not necessarily be exactly in the middle of \(a\) and \(b\) but has to lay between them. The triangular distribution is often applied having three estimates: A pessimistic \((b)\), an optimistic \((a)\) and a realistic \((c)\). The cumulative distribution function can be described as follows:

\[
F(X|a, b, c) = \begin{cases} 
0 & \text{if } x < a \\
\frac{(x-a)^2}{(b-a)(c-a)} & \text{if } a \leq x \leq c \\
1 - \frac{2(b-x)^2}{(b-c)(b-c)} & \text{if } c \leq x \leq b \\
1 & \text{if } b < x 
\end{cases}
\]  

Where:
- \(x\) Random variable
- \(a\) lower boundary
- \(b\) upper boundary
- \(c\) mode

**Normal distribution**

The normal, or Gaussian distribution, is a common applied, symmetric probability function. The central limit theory states that a large number of independent random variables drawn from a certain distribution, is distributed approximately normally. Therefore it is often applied to present an physical error or uncertainty.

\[
F(X|\mu, \sigma) = \Phi\left( \frac{X - \mu}{\sigma} \right) = \Phi(U) = \int_{-\infty}^{U} \frac{1}{\sqrt{2\pi}\sigma_x} \exp\left(-\frac{X^2}{2}\right) dX
\]  

Where:
- \(x\) Random variable
- \(\mu\) Mean value
- \(\sigma\) Standard deviation

**Exponential distribution**

The exponential distribution describes the so called Poisson process where one event continuously and independently happens. For example the event that someone enters a store. The elapsing time between the two customers can be described with an exponential distribution \((D.16)\). As the mean value of the exponential distribution is \(E(X) = \frac{1}{\lambda}\), one is able to estimate the rate parameter relatively easy.

\[
F(X|\lambda) = \begin{cases} 
0 & \text{if } x < 0 \\
1 - \exp\left(-\lambda X\right) & \text{if } x \geq 0 
\end{cases}
\]  

Where:
- \(x\) Random variable
- \(\lambda\) Rate parameter

**Gumbel-max distribution**

The Gumbel-max distribution is one of the extreme value distributions and given by equation \((D.17)\). Other often applied extreme value distributions are the Weibull or the log-normal distribution and are often applied when one has to deal with extreme values at water levels, discharges, earthquakes, etc.

\[
F(X|\mu, \beta) = \exp\left(-\exp\left(-\frac{X - \mu}{\beta}\right)\right)
\]  

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Where:

\(X\) Random variable
\(\mu\) Mode parameter
\(\beta\) Scale parameter
### D.5. Yearly maximum discharge data

Table D.15: Yearly maximum discharge data at Borgharen Dorp retrieved from live.waterbase.nl

<table>
<thead>
<tr>
<th>Year</th>
<th>Discharge $[m^3/s]$</th>
<th>Year</th>
<th>Discharge $[m^3/s]$</th>
<th>Year</th>
<th>Discharge $[m^3/s]$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1925</td>
<td>3.175</td>
<td>1985</td>
<td>1.682</td>
<td>1974</td>
<td>1.253</td>
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<tr>
<td>1994</td>
<td>2.750</td>
<td>2010</td>
<td>1.613</td>
<td>1912</td>
<td>1.243</td>
</tr>
<tr>
<td>2002</td>
<td>2.731</td>
<td>1967</td>
<td>1.590</td>
<td>1918</td>
<td>1.243</td>
</tr>
<tr>
<td>2001</td>
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<td>1963</td>
<td>1.588</td>
<td>1931</td>
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<tr>
<td>2003</td>
<td>2.390</td>
<td>1984</td>
<td>1.584</td>
<td>1976</td>
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<tr>
<td>1983</td>
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<td>1926</td>
<td>1.582</td>
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<tr>
<td>1992</td>
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<td>1919</td>
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<td>1947</td>
<td>1.575</td>
<td>1941</td>
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</tr>
<tr>
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<td>1969</td>
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<td>1946</td>
<td>1.527</td>
<td>1982</td>
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<td>1939</td>
<td>2.147</td>
<td>2012</td>
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<td>1929</td>
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<td>1979</td>
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<td>1938</td>
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<td>1977</td>
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<td>1996</td>
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<td>1924</td>
<td>2.086</td>
<td>1956</td>
<td>1.448</td>
<td>1962</td>
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<tr>
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<td>2007</td>
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<td>2006</td>
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<tr>
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<td>1972</td>
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<tr>
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<tr>
<td>1957</td>
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<td>1997</td>
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<tr>
<td>1952</td>
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<td>1954</td>
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<td>2009</td>
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</tr>
<tr>
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<td>1914</td>
<td>1.364</td>
<td>1970</td>
<td>944</td>
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<td>1965</td>
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<td>1978</td>
<td>1.364</td>
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<tr>
<td>1917</td>
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<td>1922</td>
<td>1.339</td>
<td>1959</td>
<td>809</td>
</tr>
<tr>
<td>1955</td>
<td>1.863</td>
<td>1932</td>
<td>1.338</td>
<td>1943</td>
<td>759</td>
</tr>
<tr>
<td>1998</td>
<td>1.863</td>
<td>1911</td>
<td>1.333</td>
<td>1968</td>
<td>758</td>
</tr>
<tr>
<td>1990</td>
<td>1.843</td>
<td>1964</td>
<td>1.317</td>
<td>1995</td>
<td>754</td>
</tr>
<tr>
<td>1945</td>
<td>1.744</td>
<td>1928</td>
<td>1.315</td>
<td>1975</td>
<td>697</td>
</tr>
<tr>
<td>1913</td>
<td>1.733</td>
<td>1927</td>
<td>1.309</td>
<td>1953</td>
<td>678</td>
</tr>
<tr>
<td>1916</td>
<td>1.716</td>
<td>1942</td>
<td>1.284</td>
<td>1920</td>
<td>669</td>
</tr>
<tr>
<td>1961</td>
<td>1.714</td>
<td>1949</td>
<td>1.282</td>
<td>1948</td>
<td>627</td>
</tr>
<tr>
<td>1930</td>
<td>1.712</td>
<td>1958</td>
<td>1.276</td>
<td>1933</td>
<td>553</td>
</tr>
<tr>
<td>1951</td>
<td>1.706</td>
<td>1988</td>
<td>1.275</td>
<td>1971</td>
<td>506</td>
</tr>
</tbody>
</table>
D.6. Matlab scripts

Here two matlab-scripts are presented. This is the main script corresponding for alternatives within case study Arcen:

clear all; close all; clc;

% Made bij Bart Broers
% MSc. Thesis TU Delft
% 2014-07-23
% Assumptions and limitations are formulated in the MSc Thesis report
% tic;
% % Start Oetsettings to perform the Monte Carlo simulations with Open Earth tool
% oetsettings;

%% INPUT
Input.name = 'Section 1: Alternative Ca'; %Include alternative name
Input.date = date;

%% Ring leoee input
Input.L_tot = 5000; % Length of the total ring levee
Input.L_sec1 = 100; % Length of the analyzed ring levee section
Input.L_sec2 = 550; % Length of the analyzed ring levee section
Input.L_rest = Input.L_tot - Input.L_sec1 - Input.L_sec2; % rest length

Input.Pf_old = 25; % Probability of failure due to overtopping in the old site
Input.Pf_S = [25 250 2500 25000];
Input.h_250 = 17.3732;

Input.I_c_0 = Input.L_sec1 * 4841.89; % Required height at t=0
Input.I_c_0 = Input.L_sec1 * 646.71;
Input.I_v_0 = Input.L_sec1 * 518.00;
Input.I_v_0 = Input.L_sec1 * 50

Input.I_c_m = Input.L_sec1 * 4000; % Average investments of the ring-levee
Input.I_v_m = Input.L_sec1 * 595.61; % Investments variable at t=50

Input.I_c_m = Input.L_sec1 * 464.71; % Investments constant at t=50
Input.I_v_m = Input.L_sec1 * 0; % Preventive investment constant at t=M

Input.I_c_m = Input.L_sec1 * 250; % Probability of failure due to overtopping after t=50
Input.I_v_m = Input.L_sec1 * 120; % Investment variable

%% Benefit input
Input.Pf_0 = 250; % Probability of failure due to overtopping after t=0
Input.Pf_50 = 1250; % Probability of failure due to overtopping after t=50

%% Expected damage data for interpolate/extrapolate
Ds.D = [40 65 90 140]; % Million Euro
Ds.Pf_S = [25 250 2500 25000];

%% Gambel max distribution parameters
mu = 1276.1;
sigma = 416.44;

%% Number of Monte Carlo simulations
Input.v1 = 6000;

%% Number of steps and maximum height input
Input.v2 = 11;
Input.v3 = 0.1;

%% LCCA input
Input.r = 0.025; % Real discount rate
Input.g = 0.004; % Economical growth
Input.T = 100; % Analysis time
Input.T_m = 50; % Time major maintenance

%% Pf-h relation input
% This part uses water level data in combination with corresponding pf’s.
% Water levels are used to calculate the benefits from. The water levels in
% vector hNAP correspond to the annual probability of failure: Vector Pf.
Ds.Q = [1627 1987 2260 2533 2685 2865 3008 3109 3431 4000];
% Make empty vectors and startvalues
Ds.v = zeros(Input.v1,Input.v2);
Ds.I_0_tot = zeros(Input.v1,Input.v2);
Ds.I_tot = zeros(Input.v1,Input.v2);
Ds.B2 = zeros(Input.v1,Input.v2);
A=zeros(Input.v1,1); % This vector will be used as tool to form the matrix Ds.v

delta_h1 = 0; % Startvalue of delta_h1 (above the h_min)

% Optimalization tool inclusive the Monte Carlo simulation
for j = 1:Input.v2
  % Within this loop one step of the Monte Carlo simulation is performed
  for n = 1:Input.v1
    % Monte Carlo simulation script HECU (Open earth tool)
    A(n,:) = ans.Output.z; % put the result (z) of the MC into vector A [m]
    if A(n,:) > delta_h1; % it is not allowed to have a negative levee increase
      A(n,:) = A(n,:) - delta_h1;
    else
      A(n,:) = 0;
    end
  end
  % Next the hydraulic boundary conditions of the MC simulation are stored
  % in matrix v. Hereafter vector A is used again to form the next row in
  % matrix v with the next step of delta_h1
  Ds.v(:,j) = A; %

% INVESTMENTS
% This vector calculate the total investments at t=0
Ds.I_0_tot(:,j) = Input.I_c_0 + Input.I_c_rest + Input.I_c_sec2 ... + delta_h1*(Input.I_v_0 + Input.I_v_rest + Input.I_v_sec2); % [Euro]

% This vector calculate the total investments at t=T_m
if A <= 0; % if the levee is heigh enough:
  if Input.I_p_m <= 0; % and if the preventive maintenance investments is zero,
    % no major maintenance investments have to be % taken.
    Ds.I_m_x(:,j) = 0;
  else
    Ds.I_m_x(:,j) = Input.I_c_m + Input.I_c_rest + Input.I_c_sec2 ... +(Ds.v(:,j)).*(Input.I_v_rest + Input.I_v_sec2); % [Euro]
    % Include only delta_h1 together with Ds.v(:,j) if the % existing construction have to be renewed, too. (example: glass % wall system: the whole glass wall should be replaced)
  end
end
% Total major maintenance investments
Ds.I_m_tot(:,j) = sum(Ds.I_m_x(:,j)) / Input.v1 ... /(1+Input.r-Input.g)^Input.T_m; % [Euro]

% This vector shows the total investments during the whole LOCA
Ds.I_tot(:,j) = Ds.I_0_tot(:,j) + Ds.I_m_tot(:,j); % [Euro]

% BENEFITS
% RISK R0
% First start with a vector starting from the treshold Pf_0ld to
% a phical maximum of 4600 m^3/s
Q_0 = interp1(Ds.Pf,Ds.Q,Input.Pf_0ld), 'pchip', 'extrap'); % Discharge related to Pf
delta_R = 1; % step size for the Riemann integration in the weibull distribution
Ds.v.Q_0 = [Q_0; delta_R:4600]; % Vector with treshold
Ds.v.P0 = interp1(Ds.Q,Ds.Pf,(Ds.v.Q_0), 'linear', 'extrap'); % Corresponding Pf's
% Gumbel max distribution obtained from data waterbase and easifit
z_0 = (Ds.v.Q_0 - mu)/sigma;
Gumbel_max_0 = 1/sigma * exp(-z_0 -exp(-z_0));

% Gives the resulting probability for each Riemann step
% Expected damage times probability
Ds.R0_1 = sum((interp1(Ds.Pf_S,Ds.D,(Ds.v_P0), 'linear', 'extrap')) .* (Gumbel_max_0*delta_R) *1e6));
% [Euro]

% RISK R1 (delta_h1)
PF_1 = interp1(Ds.h_NAP,Ds.Pf,(Ds.h_250 + delta_h1), 'spline', 'extrap');
Q_1 = interp1(Ds.Q,Ds.Pf,(Ds.Q_1), 'linear', 'extrap');
Ds.P1 = interp1(Ds.Q,Ds.Pf,(Ds.v_Q_1), 'linear', 'extrap');

% Corresponding Pf's
Ds.P1 = interp1(Ds.Q,Ds.Pf,(Ds.v_Q_1), 'linear', 'extrap');
Ds.P2 = interp1(Ds.Q,Ds.Pf,(Ds.v_Q_2), 'linear', 'extrap');

% Gumbel max distribution
z_1= (Ds.v_Q_1 - mu)/sigma;
Gumbel_max_1 = 1/sigma * exp(-z_1 -exp(-z_1));

Ds.R1(:, j) = sum((interp1(Ds.Pf_S,Ds.D,(Ds.v_P1), 'linear', 'extrap')) .* (Gumbel_max_1*delta_R) *1e6));
% [Euro]

% RISK R2 (delta_h1,Ds.v)
for k=1:Input.v1
PF_2 = interp1(Ds.h_NAP,Ds.Pf,(Ds.h_250 + Ds.v(k, j) + delta_h1), 'spline', 'extrap');
Q_2 = interp1(Ds.Q,Ds.Pf,(Ds.Q_2), 'linear', 'extrap');
Ds.P2 = interp1(Ds.Q,Ds.Pf,(Ds.v_Q_2), 'linear', 'extrap');

% Corresponding Pf's
z_2= (Ds.v_Q_2 - mu)/sigma;
Gumbel_max_2 = 1/sigma * exp(-z_2 -exp(-z_2));
E_D2 = (interp1(Ds.Pf_S,Ds.D,(Ds.v_P2), 'linear', 'extrap'))*1e6);
Ds.D2(k, j) = sum(E_D2.* Gumbel_max_2 *delta_R) ;
% [Euro/y]
end
Ds.R2(:,j)= mean(Ds.D2(:, j));

% Next the risk reduction is calculated with discounting
for t=1:Input.T
F0(t,:) = Ds.R0_1 / (1+Input.r -Input.g)^t ;
end
for t=1:Input.T_m
F1(t,:) = Ds.R1(:, j)/ (1+Input.r -Input.g)^t;
end
for t=Input.T_m:Input.T
F2(t,:) = Ds.R2(:, j) / (1+Input.r -Input.g)^t;
end

% The total benefits are
Ds.B(:, j)=sum(F0(:, 1)) - sum(F1(:, 1)) -sum(F2(:, 1));
% [Euro]

% OUTPUT
Output.delta_h1(:, j)=delta_h1; %Delta_h1 steps
Output.PV_B(:, j)= Ds.B(:, j); %PV(B)
Output.PV_I(:, j)= Ds.I_tot(:, j); %PV(I)
Output.NPV(:, j)= Ds.B(:, j) - Ds.I_tot(1, j); %Results NPV

% Reset vector A
A=zeros(Input.v1,1);
% Increase the delta_h1 step
Output.delta_h1=delta_h1+Input.v3;
end

% Transfer everything into one structure
NPV_optimum.Input=Input;
NPV_optimum.Ds=Ds;
NPV_optimum.output=Output;
close all; clc;
% Show resulting structure
NPV_optimum

% Plot results in three graphs next to each other
figure ('position',[100 100 1200 400])

subplot(1,3,1);
plot(NPV_optimum.output.delta_h1,NPV_optimum.output.PV_B, 'b', 'LineWidth',2);

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grid on;
set(gca, ’XTick’, NPV_optimum.output.delta_h1)
title(’Benefits’)
xlabel(’delta_h1 [m]’)
ylabel(’PV (B) [ ]’)  

subplot(1,3,2);
plot(NPV_optimum.output.delta_h1, NPV_optimum.output.PV_I,’g’, ’LineWidth’, 2);
grid on;
set(gca, ’XTick’, NPV_optimum.output.delta_h1)
title(’Investments’)
xlabel(’delta_h1 [m]’)
ylabel(’PV (I) [ ]’)  

subplot(1,3,3);
plot(NPV_optimum.output.delta_h1, NPV_optimum.output.NPV,’r’, ’LineWidth’, 2);
grid on;
set(gca, ’XTick’, NPV_optimum.output.delta_h1)
title(’NPV’)  
xlabel(’delta_h1 [m]’)
ylabel(’NPV [ ]’)  
toc  

This script represent the alternatives for case study Venlo:

clear all; close all; clc;  
%% Made bi) Bart Broers  
% MSc. Thesis TU Delft  
% 2014-08-19  
% Assumptions and limitations are formulated in the MSc Thesis report  
% tic;  
% Start Oetsettings to perform the Monte Carlo simulations with Open Earth tool  
% oetsettings;  
% INPUT  
Input.name = ’Section 3: Alternative Ba’; %Include alternative name  
Input.date=date;  

% Ring - levee input  
Input.L_tot = 7500; % Length of the total ring levee [m]  
Input.L_sec3 = 170; % Length of the analysed ring levee section [m]  
Input.L_sec4 = 130; % Length of the analysed ring levee section [m]  
Input.L_green = 1741; % rest length [m]  
Input.L_struc = 693; % rest length [m]  
Input.L_green_T_m = 2569; % rest length [m]  
Input.L_sec4 = 130; % Length of the analysed ring levee section [m]  
Input.Pf_0 = 250; % Probability of failure due to overtopping 1 [-]  
Input.Pf_1 = 178; % Probability of failure due to overtopping 1 [-]  
Input.Pf_2 = 846; % Probability of failure due to overtopping 1 [-]  
Input.Pf_3_4 = 178; % Probability of failure due to overtopping 1 [-]  
Input.Pf_50 = 1250; % Probability of failure due to overtopping 1 [-]  
Dh_h = 18.77; % Required height at t=0 [m N.A.P.]  

% Investments input:  
Input.I_c_0 = 1177.91; % Investments constant at t=0 [Euro]  
Input.I_v_0 = 677.89; % Investments variable at t=0 [Euro/m]  
% Nog even uitgaande van een lineare functie:  
Input.I_c_m = 0; % Investments constant at t=50 [Euro]  
Input.I_v_m = 677.89; % Investments variable at t=50 [Euro/m]  

% Average investments of the ring - levee  
Input.I_c_sec4 = 4000; % Investments constant at t=0 [Euro]  
Input.I_v_sec4 = 820; % Investments variable at t=0 [Euro]  
Input.I_c_green = 2210; % Investments constant whole levee [Euro]  
Input.I_v_green = 1120; % Investments variable whole levee [Euro]  
Input.I_v_struc = 820; % Investments variable whole levee [Euro]  

% Benefit input:  
Input.Pf_0 = 250; % Probability of failure due to overtopping after t=0 [-]  
Input.Pf_50 = 1250; % Probability of failure due to overtopping after t=50 [-]  

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% Expected damage data for interpolate/extrapolate
Ds.D=[70 330 595 980]; % [Million Euro]
Ds.PF_S=[125 1250 12500 125000]; %
% Gumbel max distribution parameters
mu= 1276.1 ;
sigma= 416.44 ;
% Number of Monte Carlo simulations
Input.v1=6000; %
% Number of steps and maximum height input
Input.v2=11; % Number of steps delta_h1
Input.v3=0.1; % Stepsize of delta_h1 [m]
% Real discount rate
Input.r=0.025 ; % Real discount rate [*100%]
% Economical growth
Input.g=0.004 ; % Economical growth [*100%]
Input.T=100; % Analysis time [years]
Input.T_m=50; % Time major maintenance [years]
% Pf-h relation input
% This part uses water level data in combination with corresponding pf's.
% Water levels are used to calculate the benefits from. The water levels in
% vector h_NAP correspond to the annual probability of failure: Vector Pf.
Ds.Q = [1627 1987 2260 2533 2685 2786 2865 3008 3109 3431 4000];
Ds.Pf_S=
% Make empty vectors and startvalues
Ds.v= zeros(Input.v1 ,Input.v2 );
Ds.I_0_tot= zeros(Input.v1 ,Input.v2 );
Ds.I_tot= zeros(Input.v1 ,Input.v2 );
Ds.B2 = zeros(Input.v1 ,Input.v2 );
Ds.D2= zeros(Input.v1 ,Input.v2 );
A=zeros(Input.v1 ,1);
% Startvalue of delta_h1 (above the h_min)
delta_h1=0;
% Startvalue analysis time [years]
% Optimalization tool inclusive the Monte Carlo simulation
for i=1:Input.v2
%Within this loop one step of the Monte carlo simulation is performed
for n=1:Input.v1
HBCU; %Monte carlo simulation script HBCU (Open earth tool)
A(n,:)=ans.Output.z; %put the result (a) of the MC into vector A [m]
if A(n,:) > delta_h1; %It is not allowed to have a negative levee increase
A(n,:) = A(n,:) + delta_h1;
else A(n,:) = 0;
end
end
% Next the hydraulic boundary conditions of the MC simulation are stored
% in matrix v. Hereafter vector A is used again to form the next row in
% matrix v with the next step of delta_h1
Ds.v(:,j)= A; %
% Interpolate the correct height of the current most safest structural
% and green parts
Ds.h_Pf_old_1 = interp1(Ds.Pf ,Ds.h_NAP,(Input.Pf_old_1), 'spline', 'extrap'); % [m N.A.P.]
Ds.h_Pf_old_2 = interp1(Ds.Pf ,Ds.h_NAP,(Input.Pf_old_2), 'spline', 'extrap'); % [m N.A.P.]
% INVESTMENTS
% This vector calculate the total investments at t=0
if (Ds.h_250 + Ds.v(:,j) + delta_h1) <= Ds.h_Pf_old_2
Ds.I_0_tot(:,j) = Input.I_c_0 * Input.L_sec3 +
+ Input.I_c_green * Input.L_green +
+ Input.I_c_sec4 * Input.L_sec4 +
+ Input.I_c_struc * Input.L_struc +
+ delta_h1*...
(Input.I_v_0* Input.L_sec3 +
+ Input.I_v_sec4*Input.L_sec4...
Hydraulic boundary conditions

% This vector calculate the total investments at \( t = T_m \)
if \( A <= 0 \); % if the levee is heigh enough:
   if Input.I_p_m <= 0; % and if the preventive maintenance investments is zero, % no major maintenance investments have to be % taken.
      Ds.I_m_x(:, j) = 0;
   else
      Ds.I_m_x(:, j) = Input.I_p_m;
   end
else
   if (Ds.h_250 + Ds.v(:, j) + delta_h1) <= Ds.h_Pf_old_2
      Ds.I_m_x(:, j) = Input.I_c_m *Input.L_sec3 +...
         Input.I_v_m * Input.L_sec3 * Ds.v(:, j) +...
         Input.I_v_sec4 * Input.L_sec4 *Ds.v(:, j)+...
         Input.I_v_green * Input.L_green * Ds.v(:, j)+...
         Input.I_v_struc * Input.L_struc * Ds.v(:, j);
   else
      Ds.I_m_x(:, j) = Input.I_c_m *Input.L_sec3 +...
         Input.I_v_m * Input.L_sec3 * Ds.v(:, j) +...
         Input.I_v_sec4 * Input.L_sec4 *Ds.v(:, j)+...
         Input.I_v_green * Input.L_green * Ds.v(:, j)+...
         Input.I_v_struc * Input.L_struc * Ds.v(:, j);
   end
end

% Include only delta_h1 thergether with Ds.v(:, j) if the % existing construction have to be renewed, too. (example: glass % wall system: the whole wall should be replaced)

end

% Total major maintenance investments
Ds.I_m_tot(:, j) = sum(Ds.I_m_x(:, j)) /Input.v1...
/(1+Input.r -Input.g)^Input.T_m; %

% This vector shows the total investments during the whole LCCA
Ds.I_tot(:, j) = Ds.I_0_tot(:, j) + Ds.I_m_tot(:, j); %

% BENEFITS

% RISK R0
% First start with a vector starting from the thresold Pf_old to

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% infinity (practical a discharge of 6000)
Q_0 = interp1(Ds.Pf, Ds.Q, (Input.Pf_old_1), 'pchip', 'extrap'); %Discharge related to Pf
delta_R = 0.1; % step size for the Riemann integration in the weibull distribution
Ds.v_Q_0 = [Q_0:delta_R:4600]; % Vector with threshold
Ds.v_P0 = interp1(Ds.Q, Ds.Pf, (Ds.v_Q_0), 'linear', 'extrap'); % Corresponding Pf's

% Gumbel max distribution obtained from data waterbase and easifit
z_0 = (Ds.v_Q_0 - mu)/sigma;
Gumbel_max_0 = 1/sigma * exp(-z_0 -exp(-z_0));

% Gives the resulting probability for each Riemann step
Ds.R0 = sum((interp1(Ds.Pf_S, Ds.D, (Ds.v_P0), 'linear', 'extrap').* (Gumbel_max_0*delta_R)) *1e6));

% Next the risk reduction is calculated with discounting
for t=1:Input.T
    F0(t,:) = Ds.R0 ./ (1+Input.r -Input.g)^t;
end
for t=1:Input.T_m
    F1(t,:) = Ds.R1(:,j) / (1+Input.r -Input.g)^t;
end
for t=Input.T_m:Input.T
    F2(t,:) = Ds.R2(:,j) / (1+Input.r -Input.g)^t;
end

% The total benefits are
Ds.B(:,j)=sum(F0(:,1)) - sum(F1(:,1)) - sum(F2(:,1));

% OUTPUT
Output.delta_h1(:,j)=delta_h1; %Delta_h1 steps [m]
Output.PV_B(:,j)= Ds.B(:,j); %PV(B) [Euro]
Output.PV_I(:,j)= Ds.I_tot(:,j); %PV(I) [Euro]
Output.NPV(:,j)= Ds.B(:,j) - Ds.I_tot(:,j); %Results NPV [Euro]

% Reset vector A
A=zeros(Input.v1,1);
% Increase the delta_h1 step
delta_h1=delta_h1+Input.v3;
end

% Transfer everything into one structure
NPV_optimum.Input=Input;
NPV_optimum.Ds=Ds;
NPV_optimum.output=Output;

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close all; clc;

%Show resulting structure
NPV_optimum

%Plot results in three graphs next to each other
figure('position',[100 100 1200 400])

subplot(1,3,1);
plot(NPV_optimum.output.delta_h1, NPV_optimum.output.PV_B, 'b', 'LineWidth', 2);
grid on;
set(gca, 'XTick', NPV_optimum.output.delta_h1)
title('Benefits')
xlabel('delta_h1 [m]')
ylabel('PV (B) [ ]')

subplot(1,3,2);
plot(NPV_optimum.output.delta_h1, NPV_optimum.output.PV_I, 'g', 'LineWidth', 2);
grid on;
set(gca, 'XTick', NPV_optimum.output.delta_h1)
title('Investments')
xlabel('delta_h1 [m]')
ylabel('PV (I) [ ]')

subplot(1,3,3);
plot(NPV_optimum.output.delta_h1, NPV_optimum.output.NPV, 'r', 'LineWidth', 2);
grid on;
set(gca, 'XTick', NPV_optimum.output.delta_h1)
title('NPV')
xlabel('delta_h1 [m]')
ylabel('NPV [ ]')
toc
Life-Cycle Cost Analysis

A literature study to Life-Cycle Cost Analysis, or LCCA, has been carried out. The intention of this study is to investigate LCCA for levee alternatives to achieve information regarding how and why one prefer LCCA and its methodology. The analysis approach will be introduced via a common term “project” (e.g. The Life cycle of a project is...) instead of specifying it to a design of a levee.

E.1. Introduction to LCCA

LCCA is a widely acknowledged concept as a decision tool originating from the early 1960's. The U.S. Departement of Defence initially applied it to increase effectiveness related to longer planning and to save more costs. Nowadays, the concept is a commonly applicable approach in infrastructural, architectural and machinery fields (Jawad and Ozbay 2006). The concept can be used to evaluate alternatives during a design process. In order to define LCCA, two published handbooks are cited:

“Life-cycle cost analysis (LCCA) is an economic method of project evaluation in which all costs arising from owning, operating, maintaining, and ultimately disposing of a project are considered to be potentially important to that decision.”
(The National Institute of Standard and Technology (NIST), USA, Handbook 135 (Fuller and Petersen 1996)).

“Life Cycle Cost Analysis (LCCA) is an economic evaluation technique that determines the total cost of owning and operating a facility over period of time.”
(The State of Alaska, Department of Education & Early Development (Mearig et al. 1999)).

Various approaches of this broad concept has been investigated with a vast amount of research in the last few decades (Durairaj et al. 2002). The basic principle of each literature is equal although differences occur in subheading costs, calculation of present and future values and the implementation of discount rates and inflation. Before describing different methodologies, or strategies of LCCA, a breakdown into three variables is made: Costs, analysis period and benefits.

Typical benefits of LCCA are described by Langdon (2007). The most important aspects regarding levee system alternatives are:

- Transparency of future operational costs;
- ability to plan future expenses;
- improving awareness of total costs;
- ability to optimise (future) costs (e.g. decrease maintenance costs);
- evaluation of alternatives.

Furthermore there are a number of limitations in LCCA which should be known before analysing and discussing the results with other stakeholders.
• LCCA results are only estimates;
• it require a high volume of data;
• data should be calibrated every now and then;
• the resulting value of the LCCA is not the estimate of the total costs made after its end of life;
• it can be time consuming to execute a LCCA with respect to other alternative analysis methods.

The process of LCCA is the sequence of steps taken to complete the LCCA. According to the NIST handbook (Walls and Smith 1998) the following steps should be run through:

1. Define problem and state objective
2. Identify feasible alternatives
3. Establish common assumptions and parameters
4. Estimate costs and times
5. Discount future costs to present value
6. Compute total LCCA and compare alternatives
7. Assess uncertainty of input data
8. Advise on the decision

A number of widely used methods are available for LCCA calculations. Each step described above has its different techniques with its pro’s and con’s depending on the type of project, methods used in the steps before and the level of accuracy one would like to have the LCCA. Paragraphs E.2 to E.5 describes the most common methods which are adaptable to a levee design.

### E.2. Cost terminology

Costs of a project can be divided by two major categories (Mearig et al. 1999): Initial Expenses, including the direct costs of a project (e.g. cost like design, construction, equipment, etc.), and Future Expenses presenting all costs incurred after occupation of the project (e.g. costs like maintenance, service, end-of-life costs, etc.). The last category enhance future values which can’t be added up straight away. Discount and inflation rates influence these values thorough the analysis period. How to deal with this difference between present and future values will be treated in paragraphs E.5.1 and E.5.2.

The main importance of evaluating alternatives regarding LCCA are the cost differences, not the absolute costs (Davis et al. 2005). Therefore, only costs which vary over the alternatives should be taken into account. However, this could lead to fictive cost values unfavourable to distribute with other stakeholders. Furthermore, changes due to a different planning of design are preferred to include relatively fast. Hence, one could exclude main costs equal in each alternative if the are clearly described in the report and are certainly not changing during the analysis (Fuller and Petersen 1996, Walls and Smith 1998). In normal circumstances flood defence systems should be budgeted beforehand. This indicate to analyse each alternative including all costs rather than accounting differences (easier to budged the alternative in a latter stage). Through this way, one exclude the possibility of entering a new alternative during the LCCA process influencing the finished alternative results.

#### E.2.1. Project costs

Project costs (sometimes referred to as first costs or initial cost) include all costs up to completion of a project. Commonly, the construction phase of a project is the largest part of the total costs. During an early design stage of a project, costs have to be estimated with contractor information, design teams and previous projects.

#### E.2.2. Maintenance costs

The maintenance costs are budgeted to keep he system running. Mainly these costs can be divided in three categories: Preventive, reactive and major maintenance (Davis et al. 2005).

Routine inspection to the project and testing the facility are constant in time. Depending on the type of alternative, these preventive working loads could differ a lot (e.g. specifying to a levee construction, a demountable system is relatively very costly to test with respect to a standard green levee which has no testing requirements).

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**Reactive** maintenance is the continuation of a problem or breakdown. It is an unforeseen problem which has to be solved right away. One tries to prevent this type as much as possible due to its uncertainty and high costs with respect to preventive maintenance. As supervisor of the levee, an optimum can be search for between the risk of a reactive maintenance and the costs of the preventive maintenance and inspection. An example which can be pinpointed as reactive maintenance cost is the repairing of a levee after vandalism.

**Major** maintenance has to be carried out when preventive and reactive costs are increased to a unfavourable level, or due to failure of the project. It refers to replacement of an element or structure which is at the end of its lifetime or does not fulfill its requirements. An upgrade to the project can also be depicted as major maintenance. A rapid increase in a river discharge could lead to higher water. Hence, a structure need to be heightened or strengthen.

**E.2.3. Service costs**
Service costs are all cost to provide the function of the project such as electricity, water and fuel. In typically LCCA these costs are not considered (e.g. buildings) (Davis et al. 2005). However, for levee design purpose, service costs could differ significantly. An huge stock to stable temporary and demountable systems is costly with respect to alternatives where the system elements are included within the design.

**E.2.4. End-of-Life costs**
At the end of the (assigned) lifetime a project could still have some value. This is called the **salvage Value**. It consist basically of two components, the residual value and the serviceable life.

The **residual value** reflects the amount of money which can be achieved by recycling the material and the cost of demolition. Usually these costs are assigned to a new project but have to be taken into account in the LCCA (Davis et al. 2005).

The **Serviceable life value** represent the remaining life of a project at the end of the analysis period. It makes a distinction between alternatives if the haven't the same end-of-life period. An alternative which need major maintenance every 45 years comparing with an alternative with major maintenance every 50 years have to be calculated with one analysis period (see paragraph E.3). For instance, if the analysis period is 90 years, the second alternative has a serviceable life value of 10 years (50∗2−90 = 10 years) (Walls and Smith 1998).

**E.2.5. Sunk costs**
Sunk costs represent a value which is irrelevant for the LCCA. In order to explain sunk costs an example from Walls , Smith & Michael, 1998 (Walls and Smith 1998) is presented below:

"An individual places a $10 non-refundable deposit on a $100 camera at Store A. Before picking up the camera, the individual finds an identical camera on sale at Store B for $80. From an economic efficiency perspective, from which store should the individual purchase the camera? What bearing does the $10 deposit have on the decision?"

It is irrelevant to the above discussion that one invested the $10 dollars beforehand. These costs are the sunk costs and should not influence the analysis to which camera will be chosen. The decision has to be based on the camera in store A with a value of $90 and the same camera in store B with a value of $80. This example shows a clear picture of the sunk costs. Often these costs are not that clear. Former investments of a levee design should not be calculated into the LCCA unless one uses the former construction. In the latter case, it will result in less project value.

**E.3. Analysis period**
The second part of the breakdown of the LCCA introduction is the analysis period. It reflect the length of time over which an LCC is analysed (Langdon 2007). Sometimes the analysis period coincide with the total life of the project while others prefer to use the time horizon of the investor (Fuller and
Petersen 1996). According to the Airfield Asphalt Pavement Technology Program (AAPTP 2011), the analysis period for LCCA must be sufficiently long such that each alternative include at least one major maintenance event. Important to emphasize is the fact that each alternative has to be calculated with the same analyse period.

In theory, the analysis period can be derived into an endless stream of cash flows. This coincide with the nHWBP. Practically an often applied period of 100 years, (including demolition) plus the period of construction, is used (van den Berg et al. 2013). Future expenses have to be calculated to their present value with help of a discount rate. Using a rate of 2.5% leads to less than 8.5% of the total costs made 100 years after completion. Theoretically the analysis period is infinity for levee designs, but practically this is impossible to calculate. Therefore one uses a total analysis period of 100 year after completion of the project, which is in line with the nHWBP (van den Berg et al. 2013).

E.3.1. base time
All expenses should be adjusted to the “present” value of money before adding them up to a certain time. This time is called the base time. Often it correlates at the end of the construction phase. The LCCA results are normally presented with the present-year as base time. When dealing with levees in the Netherlands, a significant period can be suspected (a few years) between designing and finishing of the construction phase. This interval is the same for all alternatives. Designing the levee in most cases is a small amount with respect to the project costs (for instance, 10% of the construction costs). Therefore the base time at the suspected construction period should be used which might not coincide completely to the actual completion of the levee.

E.4. Benefits
Different methods are available how to assign and calculate benefits. Literature describes methods such as the Payback period (PB), Net Present Value (NPV), Net Benefit (NB), Savings to Investment Ratio (SIR), etc. (Langdon 2007). In many cases during a LCC analysis one can include those benefits relatively easily. This is illustrated with a small example:

You are considering to buy a new motor for your car which reduces the expenditure of fuel per driven kilometre. besides the direct purchase costs, maintenance and fuel have to be included into the LCCA as well. Secondly, you estimated the following numbers:

- Total lifetime of the motor is 10 years;
- you drive on average 20.000 kilometres per year;
- the expenditure of fuel per kilometres is €0,01;
- purchasing costs of the motor are €500,-;
- total maintenance in 10 years cost €300,-.

Calculating future expenses to the present value and the future change of fuel costs are neglected in this example. The total investment of the motor will be: €800 (excluding the costs of the fuel). The benefits in this example are 10 * 20.000 * €0.01 = €1000. The investments with these estimates is beneficial.

Defining benefits for flood defence systems is more difficult as some values are hard to determine. for instance, expressing the exact benefits due to a flood is much more subtle and discussable.

Cost/benefit analysis methods are presented as an useful instrument for decision-making on a desired flood protection strategy in the Netherlands (Jonkman et al. 2004). They are applicable to create safety standards which implement criteria how high and strong a levee should be.

E.4.1. Cost/benefit analysis of a flood protection system
A flood protection system is never 100% absolute safe as there will always be a certain flood risk. The main socio-political idea is the consideration of which risks are acceptable and which ones are not. The basic principle to determine the acceptable risk requires that a project has an increase in economic welfare. This can be done by analysing the investment costs of a levee versus the decrease
in potential flood damage of the hinterland. One could speak of economic welfare if a measure reduces the economical damage more than it actually costs.

Different types of costs have to be included to account for a levee measure: Project costs, maintenance costs, service costs and End-of-life costs.\(^1\) Although levee safety standards are based upon cost/benefit analyses, the most cost-effective design could differ in failure probability. In these cases the levee design should primarily be based on the safety requirements rather than the levee design with the optimal LCC.

Besides discounting future expenses to their present value one should include thee economical growth as well. The estimated flood damage could increase due to new buildings and more citizens.

**E.4.2. Benefits and LCCA**

Three different proposals are made how to deal with benefits for levee design alternatives including the knowledge of the safety standards: Payback period, Net present value calculation or not including benefits.

**E.4.3. Payback period**

The Payback period is a value how long it takes before an investment is paid back. In the example mentioned earlier in this paragraph the PB will be 8 years. The calculation of the PB can be one of the output parameters to discuss alternatives with other stakeholders (e.g. we do not know what the river Meuse will do in the future, one would like to chose between alternatives how long it takes before they are paid back. It might be the case that a levee system has to be adjusted after 20 years. An alternative with a PB less than 20 years might be preferable).

**E.4.4. Not including benefits**

In this approach one use the criteria from the safety standards. Some alternatives may have more robustness than others but this is not taken into account in the LCCA. The client is able to chose an alternative (which might be more expensive) above an other one due to its robustness.

**E.5. Net Present Value**

The Net Present Value (NPV) is one of the most widely used criterion in order to evaluate life-cycle costs of alternatives (Langdon 2007). Cost of a project occur at different point in time each with its values at that moment. In order to combine these costs with the direct costs the have to be discounted into present values by use of so called discount rate. This discount rate depends on the investment rate(s) and sometimes include inflation or deflation (Krützfeldt. 2012), both described in paragraph E.5.1 and E.5.2. It is essential that the same discount rate and inflation treatment will be used in LCCA of multiple project alternatives (Fuller and Petersen 1996). By including benefits of an alternative, the NPV can be calculated as indicated in E.1. A project is beneficial as long as the NPV is positive, ideal to analyse if a project should be carried out at all.

\[
NPV = PV(B) - PV(I)
\]  

(E.1)

Where:

- \(NPV\) Net Present Value [€]
- \(PV\) Present Value [€]
- \(B\) Benefits of flood risk reduction [€]
- \(I\) Investments [€]

**E.5.1. discount rate**

When choosing among potential projects or alternatives, one has to account the timing of the cash flows. Costs occurring at different points in time have other values. Spending 100 Euro today is more expensive versus 100 Euro after one year. The interest rate is responsible for this phenomena. If for instance one saves the 100 Euro on a bank account with an interest rate of 2%, the next year it will be 102

\(^1\) (Jonkman et al. 2004) only defined the project-, maintenance- and service (or management)-costs.

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Euro. Hence, an investor prefers cash receipts later. The above example is presented in equation (E.2).

\[ C_1 = C_0 + i \times C_0 = C_0 \times (1 + i) \quad \text{(E.2)} \]

Where:
- \( C_0 \) Cash receipt at \( t=0 \)
- \( C_1 \) Cash receipt at \( t=1 \)
- \( i \) Interest rate (in example 2%)

If one would calculate for a multiple number of years, each with the same interest rate \( i \), the multiplication factor can be used \( t \) times.

\[ C_t = C_0 (1 + i)^t \quad \text{(E.3)} \]

Where:
- \( C_t \) Cash receipt at time \( t \)
- \( t \) Period of study time since the base time \( t=0 \)

Future cash flow must be discounted to their present value (see paragraph E.3) before combining them into the LCCA. This is illustrated with equation (E.4). \( PV(C) \) is called the present value of the total costs and the interest rate \( i \) is changed in the discount rate \( d \).

\[ PV(C) = \frac{C_0}{(1 + d)^t} \quad \text{(E.4)} \]

The discount rate is used to change future cash flows to present values based on the time of the cash flow. Note that the type of investor (private or public) could have different investments opportunities which result in a significant variation in discount rate. The NIST Handbook (Fuller and Petersen 1996) defines two methods regarding the discount rate:

1. Real discount rate (using constant values);
2. Nominal discount rate (using current values).

Each method ends up with the same results (if all rates are constant over time) but have its own advantages. Nevertheless, equation E.4 can be used in both methods. The discount rate is an important and often controversial piece of the LCCA process, because it can influence the results of the analysis significantly (AAPTP 2011).

Generally speaking, Dutch practice is to use a real discount rate of 5.5%. This include the 2.5% risk free real discount rate and additionally a 3% discount rate accounting for macro-economic risks (van den Berg et al. 2013). The probability of economic negative developments are weighted higher in contrast to the probability of positive developments (RWS 2012). Costs and benefits that arise after a few decades hardly count, while long term effects play a major role on levee alternatives. In contrast to the Dutch government, The United Kingdom switched from a fixed discount rate over time to one that decreases over time. The first 30 year a real discount rate of 3.5 % is assigned while the years between 31 and 75 a discount rate of 3.0% will be used. This continuously decreases up to 1 % after a period of 300 years. France also uses a variable discount rate over time. Most literature describes real discount rate around the 3-5 % which coincide the present Dutch practise (Langdon 2007, Davis et al. 2005).

One of the key questions is whether the discount rate should vary or stay constant over time. The delta programme of 2015 therefore will state this into more detail based on results of the CBP (Netherlands Bureau for Economic Policy Analysis) study (DP2014 2013). For this LCCA of levees a real discount rate of 2.5 % will be used. A check will be made with the use of other rate to analyse the dependency of the discount rate onto the levee design alternative analysis.

### E.5.2. Inflation or deflation

Inflation and deflation equalize the value of purchasing power of money over time. Inflation is the result of an increase in the amount of money balanced with the economic productivity. A smaller productivity in ratio would lead to an increase in average price of products and services as no additional

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production is present. Due to this price, the purchasing power would drop down (One can buy less products/services with the same amount of money). The idea is that inflation would equalize one’s purchasing power.

To include inflation/deflation into the present value calculation (see, paragraph E.5), there are two ways to do so related to the two methods described earlier. If products or services are presented in the particular year the costs are expected, one speaks of current value. Hence, the current-value amount is the money of any one year's purchasing power inclusive inflation. In contrast, the amount of money spend with a uniform purchasing power is called the constant value which is exclusive inflation. Constant value indicate how much money one need to buy the same products or services at different times. It is generally easier to conduct an economic analysis with constant value. With the current value approach, one need a study period analysis with an inflation estimation per year. The current value approach is more appropriate in the private sector analysis. There taxes are involved which need to be computed on actual cash flow (Fuller and Petersen 1996). As water boards and the general government both spend money to a levee design, one need to consider how much money is available in the future and if it reflects current or constant value.

![Figure E.1: Basic understanding of inflation. red: Total budged to spend for project, blue: budged needed for project A. Both are influenced by inflation with a rate of 1 percent. The ratio of the total budget and project A is constant](image)

To clarify inflation with its current and constant value system, figure E.1 shows two histograms. The biggest, red one indicate the total amount to invest in projects (Like a water board has its budged to spend at levees) and the smaller, blue one suggest and arbitrary project "A". The inflation rate is assumed to be 1%. In this case, it is assumed that money to invest in projects raise at the same level as the inflation. Project A can be invested after 50 years for the value at t=0 year of € 100,- which reflects the constant value. The value of € 164,- needs to be invested at the time t=50 year which reflect the current value.

E.5.3. Price escalation

Some products or services do not change exactly with the rate of general inflation or deflation. They even may have a significant difference between those rates, e.g. Fuel oil, energy or steel prices. One could suspect a larger increase of energy costs with respect to general inflation over time. The price escalation can be divided into the two approaches: Nominal price escalation and the real price escalation (Fuller and Petersen 1996).

The nominal price escalation accounts the increase of commodity at a certain time in the future. The cash receipt at the base date has to be adjusted when calculating the nominal price escalation...
(see, equation E.5).

\[ C_{t,\text{current}} = C_0(1 + E)^t \]  

(E.5)

Where:
- \( C_{t,\text{current}} \) Current cash receipt at \( t \)
- \( C_0 \) Cash receipt at \( t=0 \)
- \( E \) Nominal price escalation rate

The **real price escalation** approach uses the difference between the general inflation rate and the nominal price escalation rate. Hence, before calculating the cash receipt at time \( t \), the nominal price escalation has to be adjusted with the inflation rate (see, equation (E.7)). Next the cash receipt at time \( t \) can be calculated again with

\[ e = \frac{(1 + E)}{(1 + I)} - 1 \]  

(E.6)

\[ C_{t,\text{constant}} = C_0(1 + e)^t \]  

(E.7)

Where:
- \( e \) Real price escalation rate
- \( E \) Nominal price escalation rate
- \( I \) Inflation rate
- \( C_{t,\text{constant}} \) Constant cash receipt at \( t \)

Both approaches can be used to adjust the price escalation of a certain product or service. Often it depends on the type of data available how to include the price escalation into the NPV calculation.

**E.5.4. Example of Present Value calculation**

Paragraph E.5.1 stated two methods to determine the NPV: using a real discount rate (expressing cash flow in constant values) and using a nominal discount rate (expressing cash flow in current values). Equations (E.8) and (E.9) are showing an example to clarify those methods (Fuller and Petersen 1996).

*Example: Suppose one would like to know the present value of a structure which has to be replaced in 20 years. If the station has been replaced today, the price would be € 1000. Due to advanced manufacturing of the pumps one expects an increase of the price at a rate of 2 percent in contrast to the general price inflation. The rate of general price inflation is estimated on 4 percent per year. The discount rate is 3 percent.*

**Net Present Value calculation with Real discount rate:**

\[ PV = F_{t=0} \times \left( \frac{1 + e}{1 + d} \right)^t \]  

(E.8)

Where:
- \( PV \) Present value [€]
- \( F_{t=0} \) Value of replacement at \( t=0 \) years (example=1000) [€]
- \( e \) Price escalation rate (example=-0.02) [-]
- \( d \) Real discount rate (example=0.03) [-]

**Net Present Value calculation with Nominal discount rate:**

\[ PV = F_{t=0} \times \left( \frac{1 + E}{1 + D} \right)^t \]  

(E.9)

Where:
- \( PV \) Present value [€]
- \( F_{t=0} \) Value of replacement at \( t=0 \) years [€]
- \( I \) Inflation rate (example=0.04) [-]
- \( E \) Price escalation rate \((1 + e) \times (1 + I) - 1\) (example=0.0192) [-]
- \( D \) Nominal discount rate \((1 + d) \times (1 + I) - 1\) (example=0.0712) [-]

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At the end both approaches resulting in the same present value of € 369.64, accounting just the replacement costs after 20 year. Please notice the data concerning the price escalation with respect to inflation. In this example the real price escalation is given. Data in other circumstances could be constant or current values which should be known beforehand.

The NPV calculation with the constant value and real discount rate will be used during the LCCA of the flood defence within this research.

**E.6. uncertainty analysis**

The output of an LCCA provide important decision-making information. Possible errors within the analysis due to uncertainties are part of the LCCA. Although only one LCC is defined per project or alternative, it actually is an expected value of a distributed LCC.

Figure E.2 shown the variety of different methods and rules which can be used to calculate/indicate the possible errors due to uncertainties. Hydraulic boundary conditions are the only aspects where their uncertainties are included within this research. Other uncertainties are not inserted like the price escalation, discount rate, economical growth, material quantities, etc.

![Diagram](image)

**Figure E.2: Common tools and techniques in risk/uncertainty analysis (Langdon 2007)**

The uncertainties in hydraulic boundary conditions affect the investments as well as the benefits. The conditions are formed on a deterministic and probabilistic manner. A Monte Carlo simulation is chosen to determine the combined uncertainties in hydraulic boundary conditions. A sensitivity analysis will be carried out to perform a sensitivity analysis for these hydraulic aspects.

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Design information case study
Venlo

In order to validate the preliminary conclusions of the research, a second case study is used. This appendix state data and outcome of the "toolbox" and LCCA for case study Venlo.

F.1. General information
Currently the VNK-report of ring-levee 68 Venlo is still in development. Information like the present safety assessment and available data to calculate the benefits are collected from a concept version of this report.

F.1.1. Governing levee-section(s) and cross-section(s)
Before and during the procedure of defining levee alignment options, the levee can be divided into sections each with their similar parameters (e.g. cross-section, present construction type, space, functions, etc.). Figure F.1 illustrate which sections will be investigated during this research.

The case study has two sections; three and four (there are no sections one and two within this case study). There are two governing aspects causing enough difference to make this distinction: First there are many trees positioned in section four which might result on different realistic levee alternatives. Secondly there are future construction plans of large multilevelled apartment buildings. Levee design alternatives which coincide with these plans might differ of the alternatives favourable in section three.
Per section one governing cross-section is chosen. The governing cross sections are shown in the paragraphs F.3.1 and F.3.2.

**F.1.2. Present levee structure**

The flood defence system is placed along/above an old historical wall. It consist of an L-shaped concrete element with partly a demountable stop-log system. Figure F.2 shows the available data of a cross section in the database of the Water board.

![Cross-section Venlo DWP 68.104+19](WPM 2010)

The current demountable system does not fulfil the failure to close mechanism related to the annual probability of exceedance of 1/250 According to Slootjes and Stijnen (2007).

The ring-levee of Venlo has been separated in two parts I and II. The sections three and four laying within part I. This part has a total length of about 7500 meters, consisting of green levees (approximately 5365 meters) and structural levees (2135 meters). The lengths in section three and four are 130 and 170 meters respectively (both structural levees).

**F.1.3. Present safety assessment**

Per part, a HIS-SSM analysis has been carried out by VNK (2014). A breach in the Northern part (II) lead to only damage in this part of the ring-levee, while a breach in the south affects the whole area. In the worst case scenario multiple breaches occur in both part leading to a total expected economical damage of 1195 million Euro’s. The sections analysed in this research are located in part I.

The present annual probability of failure due to overtopping and overflow varies over the ring-levee between 1/1.500 and 1/70. The part which will be analysed further has an annual probability of failure due to overtopping and overflow of 1/1.100. However, in contrast to the case study Arcen, overflow and overtopping is not the governing failure mechanism. The annual probability of failure due to piping and heave is in some cases much larger. Subsequently, failure to closure of a demountable system should lead to a higher failure probability during the advanced statutory assessment of the ring-levee (Niemeijer *et al*. 2013, Slootjes and Stijnen 2007). This failure to close mechanism is not (yet) included in the draft report. Considering only the height would lead to a large underestimation of the benefits.

The current probability of failure is assumed to be a summation of the failure mechanisms overtopping and overflow and the estimated failure to close. The minimum total annual failure probability is 1/250. An assumptions is made to the current probability of failure of the sections 3 and 4.

Unfortunately, formulating the probability of failure of the whole ring-levee is a difficult task. The levee consist of many different and small levee elements each with a different annual probability of failure.
Therefore some assumptions are made in order to execute the optimization calculation. Table F.1 shows all levee section (levee names) assessed by the VNK with their corresponding probability of failure \( p_f \). Currently the required annual probability of exceedance is 1/250. All levees who do not fulfil this requirement will be increased/reinforced in the year 2020.

The new safety approach enhance an annual probability of failure for ring-levee 68 of 1/1.250 (Berkhof et al. 2013). All levees in table F.1 with a larger annual probability of failure will be increased/reinforced in the year 2070. The remaining levees fulfill both requirements.

Table F.1: Overview of the annual failure mechanisms and their lengths of ring-levee 68, part I (VNU 2014)

<table>
<thead>
<tr>
<th>Levee name</th>
<th>( p_f )</th>
<th>2020</th>
<th>2070</th>
<th>Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>DV01 hmp32,00-32,80</td>
<td>1/100</td>
<td>x</td>
<td>x</td>
<td>80</td>
</tr>
<tr>
<td>DV02 hmp32,80-39,13</td>
<td>1/790</td>
<td>x</td>
<td>x</td>
<td>33</td>
</tr>
<tr>
<td>DV03 hmp39,13-45,80</td>
<td>1/140</td>
<td></td>
<td>x</td>
<td>667</td>
</tr>
<tr>
<td>DV04 hmp45,80-54,00</td>
<td>1/880</td>
<td>x</td>
<td>x</td>
<td>820</td>
</tr>
<tr>
<td>DV05 hmp54,00-68,00</td>
<td>1/910</td>
<td>x</td>
<td>x</td>
<td>1,400</td>
</tr>
<tr>
<td>DV06 hmp68,00-73,90</td>
<td>1/230</td>
<td>x</td>
<td>x</td>
<td>590</td>
</tr>
<tr>
<td>DV07 hmp73,90-78,00</td>
<td></td>
<td></td>
<td></td>
<td>410</td>
</tr>
<tr>
<td>DV08 hmp78,00-85,00</td>
<td>1/200</td>
<td>x</td>
<td>x</td>
<td>700</td>
</tr>
<tr>
<td>DV09 hmp85,00-90,00</td>
<td>1/640</td>
<td>x</td>
<td></td>
<td>500</td>
</tr>
<tr>
<td>DV10 hmp90,00-93,20</td>
<td>1/720</td>
<td>x</td>
<td></td>
<td>320</td>
</tr>
<tr>
<td>DV11 hmp93,20-99,00</td>
<td>1/1,000</td>
<td>x</td>
<td></td>
<td>680</td>
</tr>
<tr>
<td>DV12 hmp99,00-101,65</td>
<td>1/1,500</td>
<td></td>
<td></td>
<td>265</td>
</tr>
<tr>
<td>DV13 hmp101,65-108,50</td>
<td>1/1,100</td>
<td>x</td>
<td></td>
<td>385</td>
</tr>
<tr>
<td>DV13 hmp101,65-108,50</td>
<td>1/200</td>
<td></td>
<td>x</td>
<td>300</td>
</tr>
<tr>
<td>DV14 hmp108,50-113,65</td>
<td>1/1,500</td>
<td></td>
<td></td>
<td>350</td>
</tr>
<tr>
<td><strong>Total length</strong></td>
<td><strong>2337</strong></td>
<td><strong>5795</strong></td>
<td><strong>7500</strong></td>
<td></td>
</tr>
</tbody>
</table>

The investments of the 2020 and 2070 levee increasement/reinforcement are calculated with help of the total influence of the green and structural levees. Hence, instead of investigating every levee part along the 7,5 kilometer levee, one estimate the green levees and structural levees in both periods by

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help of their ratio. This lead us to the following values:

\[
L_{\text{green levees,2020}} = 2.337 \times \frac{5.365}{7.500 - 300} = 1.741
\]  
\[
L_{\text{structural levees,2020}} = 2.337 \times \frac{2.135}{7.500 - 300} = 693
\]  
\[
L_{\text{green levees,2070}} = 5.795 \times \frac{5.365}{7.500 - 300} = 4.318
\]  
\[
L_{\text{structural levees,2070}} = 5.795 \times \frac{2.135}{7.500 - 300} = 1.718
\]

Subsequently, the average annual probability of failure will be calculated. Long sections contribute more than sections with a smaller length. By multiplying the probability of failures with the lengths per section and deviate them with the total length, one is able to estimate the average probability of failure. Table F.2 states an overview of the values required to perform the optimization calculation.

<table>
<thead>
<tr>
<th>Levee type/section</th>
<th>Year</th>
<th>Length [m]</th>
<th>(P_f) [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Green</td>
<td>2020</td>
<td>1.741</td>
<td>1/178</td>
</tr>
<tr>
<td>Green</td>
<td>2070</td>
<td>1.741</td>
<td>1/250 (+ P_f(Δh1))</td>
</tr>
<tr>
<td>Green</td>
<td>2070</td>
<td>2.569</td>
<td>1/846</td>
</tr>
<tr>
<td>Structural</td>
<td>2020</td>
<td>693</td>
<td>1/178</td>
</tr>
<tr>
<td>Structural</td>
<td>2070</td>
<td>693</td>
<td>1/250 (+ P_f(Δh1))</td>
</tr>
<tr>
<td>Structural</td>
<td>2070</td>
<td>1.025</td>
<td>1/846</td>
</tr>
<tr>
<td>Section 3</td>
<td>2020</td>
<td>170</td>
<td>1/200</td>
</tr>
<tr>
<td>Section 3</td>
<td>2070</td>
<td>170</td>
<td>1/250 (+ P_f(Δh1))</td>
</tr>
<tr>
<td>Section 4</td>
<td>2020</td>
<td>130</td>
<td>1/200</td>
</tr>
<tr>
<td>Section 4</td>
<td>2070</td>
<td>130</td>
<td>1/250 (+ P_f(Δh1))</td>
</tr>
</tbody>
</table>

Ring-levee 68 of Venlo consist of two parts. The first Southern part will be analysed within this research. This part consist of Green and structural parts. Alternatives will be formed for sections three and four. Depending on the average probability of failure, part of the green and structural levees will be reinforced/increased up to at least 1/250 probability of failure requirement in 2020. In 2070 levees will be reinforced to an annual probability of failure of 1/1250. Again the green and structural sections should be raised to. Therefore table F.2 shows the two times the green and structural types in 2070 (the both have a different height). The same green/structural ratio is used as found for the entire part 1 of the ring-levee.

**F.1.4. Expected damage**

During the research data is used from the conceptual report of VNK (2014) which present the results of a HIS-SSM analysis. The results are shown in table F.3 and the corresponding graph in figure 3.2.

The economical damage and casualties are estimated at four different water levels (H-1D, H, H+1D and H+2D) in where D represent the so called decimation height. H is the water level equal to the present statutory assessment. The expected damage, related to the required flood probability, can be estimated by interpolating and extrapolating these values.

The expected economical damage varies between the 1.000 a 5.000 Euro’s per hectare per year in most part of the ring-levee (VNK 2014). An assumptions is made regarding the difference in benefits due to a change in levee alignment over the alternatives. 5.000 Euro per hectare with respect to the present situation is estimated as benefit difference due to the central position, the type of buildings and the number of citizens.

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### F.2. Levee alignment options

Various levee alignment options are assigned within this case study. The options are displayed for all considered section of the ring-levee but should be analysed separately.

Due to the height difference and the small profile of the river Meuse in Venlo, no additional levee alignment option is chosen near the river-side. There is no significant advantage to this option as the view of the residents decreases, the flow profile decreases, there is enough room for a levee at the present location and construction costs are probably much larger than the other alternatives.

**Option 0: Present levee alignment**

Option 0 is shown in figure F.5. As this option lays at the same position as the present situation, it does not influence the Q4 project much, expect the available view at certain places.

**Option 1: Levee alignment at the river-side of the road**

This option, shown in figure F.6, may be most of interest in section 4. Within the project Q4 some high raised building are planned. At the moment their are uncertainties whether or not these buildings will be executed like the economical crisis resulting in a lack of possible residents. Figure F.7 shows an impression of the Q4 design.

The ground floor of the buildings are probably parking places only. The levee may be placed along or underneath the building if and only if the levee is with a great certainty robust enough, or it can be raised.

**Option 2: Change levee alignment at the land-side of the road**

At the moment, both sides of the Maaskade road have parking lots and a walking path. Changing the levee alignment such that it lays in front of the houses creates a much better view for the people.

---

#### Table F.3: Expected economical damage and number of casualties from the VNK2 analysis of the ring-levee Venlo part I (VNK 2014)

<table>
<thead>
<tr>
<th>Water levels</th>
<th>Probabilities</th>
<th>Economical damage</th>
<th>Casualties</th>
</tr>
</thead>
<tbody>
<tr>
<td>H-1D</td>
<td>1/125</td>
<td>€125 million</td>
<td>5</td>
</tr>
<tr>
<td>H</td>
<td>1/1.250</td>
<td>€405 million</td>
<td>30</td>
</tr>
<tr>
<td>H+1D</td>
<td>1/12.500</td>
<td>€740 million</td>
<td>60</td>
</tr>
<tr>
<td>H+2D</td>
<td>1/125.000</td>
<td>€1195 million</td>
<td>145</td>
</tr>
</tbody>
</table>

---

**Figure F.4: Expected flood damage in relation with the flood probability in Arcen (data used from VNK 2014)**

\[ y = 153.96 \ln(x) - 658.85 \]

\[ R^2 = 0.9876 \]
travelling. Moreover, the view from the first floor of the residents increases as well.

**option 3: Change levee alignment to the other side of the houses (Q4)**

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Perpendicular to the river, land height increases rapidly. A levee aligned at the other sides of the project Q4 might cost much less in contrast to the other options.

The unprotected area of Q4 within this levee alignment option has an area of approximately 1 hectare.
F.3. Available space

Next, all possible levee design options are gathered per levee-section and levee alignment option. With the aid of design criteria and the governing cross-sections a sketch design is made (see chapter 2). Whenever it turns out that there is not enough space available, the alternatives are not assessed further which is reported in paragraph F.3.1.

F.3.1. Ring-levee section 3

This paragraph state all alternatives of levee section 3 together with a cross-sectional sketch design. The length/height ratio of all cross-sections is 1:2.

At the river-side (left hand side) of the flood defence within the governing cross-section an access road is present. Therefore this part is not used during the alternative analysis.

Option 0: Present position; A: Green levee

Alternative A is sketched in the governing cross-section as can be seen in figure F.10.

Option 0: Present position; B: Demountable system

Currently the present demountable system fails the statutory assessment. It is uncertain if these kinds of systems will fulfill the requirements with the new safety philosophy. Therefore, it is chosen to implement a demountable system within the analysis. Part of the road/paths are raised to fulfil all requirements following from the design criteria. Figure F.11 shows the sketch of alternative B.

Option 0: Present position; C: multiple wall system

There are many ways to form a multiple-wall system as an alternative. Besides the double wall system, there is an option to include another retaining wall to create extra safety and accessibility. One can see clearly that there is enough space for these types of alternatives. The three walls system is chosen (figure F.12) as the next alternative has the same principle as a standard double-wall system.

Option 0: Present position; D: Structural design option and a partly raise of the surface
By raising the surface at the land side of the present levee, one is able to raise the structure itself with concrete. The sketch of alternative D is shown in figure F.13.
Option 0: Present position; E: Glass wall system

Alternative E (figure F.14) consist of a glass wall system on top of the present situation. The road has to be raised slightly in order to have a maximum height of 0.80 meter like indicated in the design criteria.

Option 0: Present position; F: Dynamic system with a probable small increase of the surface

One of the demountable systems are possible, too. Of course there are some uncertainties if these types of system fulfil the present and new safety requirements but are of great interest within the important area of Venlo. The dynamic systems are not considered separately as most systems have their own contractors/patents. Figure F.15 shows the dynamic system within the governing cross-section.

Option 1: Change levee position to the river-side of the road; G: glass wall system

The road and paths will be redesigned such that a glass wall system can be constructed somewhere in the middle. The new concrete part of the present levee structure will be demolished. The road/path on the river-side of alternative G can be lowered if preferred such that one experience the historical city wall more intensive. The underground infrastructural elements have to be reconstructed. A sketch is included in figure F.16.
F.3. Available space

**Figure F.15:** Option 0: The same position; F: Dynamic system with a probable small increase of the surface

**Figure F.16:** Option 1: Change levee position to the river-side of the road; G: Glass wall system

**Option 1:** Change levee position to the river-side of the road; H: Dynamic system

Instead of a glass wall, here a dynamic system is used as can be seen in figure F.17.

**Option 2:** Change levee alignment to the land-side of the road; I: Glass wall in front of houses

An extra hall is created consisting of a concrete foundation/wall and a glass wall. Occasionally a pass-through is present which can be closed by a demountable system. Valves have to be installed within the sewage system to prevent seepage. A sketch is shown in figure F.18.

**Option 2:** Change levee alignment to the land-side of the road; J: Multiple wall system

A multiple wall system is moved as much as possible to the land-side to limit the visual impact. Here changes have to be made regarding the underground infrastructure, too. A big disadvantage is the difficulty for residents to access their houses properly. Figure F.19 illustrate alternative J.

**Option 2:** Change levee alignment to the land-side of the road; K: Connect sheet pile with house protection system

Here a house protection system have to be constructed combined with a concrete floor and a sheet pile. The latter two structures limit seepage. At this moment it is unknown if these kind of systems

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Figure F.17: Option 1: Change levee position to the river-side of the road; H: Dynamic system

Figure F.18: Option 2: Change levee alignment to the land-side of the road; I: Glass wall in front of houses

Figure F.19: Option 2: Change levee alignment to the land-side of the road; J: structural design option in front of houses

will fulfil the failure mechanism "failure to close" As it consist of multiple dynamic or demountable doorways, brick-work protection elements and multiple valves. The total flood protection system have

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to be established up to a height of 2,6 meter.

**Figure F.20:** Option 2: Change levee alignment to the land-side of the road; K: Connect sheet pile with house protection system

**Option 3: Change levee alignment to the other side of the houses (Q4); L: Connect to higher grounds**
The road at the other side of the houses is high enough. When connecting this road with the present flood defences at both sides of the section the levee is finished up to major maintenance given the deterministic hydraulic boundary conditions after 50 years. Hereafter the street might have to be raised with two retaining walls.

**Figure F.21:** Option 2: Change levee alignment to the other side of the houses; L: Connect to higher grounds

**F.3.2. Ring-levee section 4**
This paragraph state all alternatives of levee section 3 together with a cross-sectional sketch design. The length/height ratio of all cross-sections is 1:2.

**Option 0: Present levee alignment; A: Green levee**
Alternative A is a green levee (parallel levee) and is sketched in the governing cross-section as can be seen in figure F.22.

**Option 0: Present levee alignment; B: Demountable system**
Currently a demountable stop-log system is already present. Future changes in the safety assessment
could lead to a positive outcome during the statutory check. Besides, including this alternative within the analysis gives insight about how realistic and expensive it is in contrast to other alternatives. During major maintenance is should be raised by buying additional equipment and reinforcing the current concrete capping beam. The sketch is included in figure F.23.

**Option 0: Present levee alignment; C: Glass wall system**
A glass wall system can be placed on the present situation relatively easily. An advantage is the limited impact on the view. However, such glass system has to be cleaned every once in awhile. Furthermore the glass system is sensitive for vandalism which has been discovered on glass walls nearby.

**Option 0: Present levee alignment; D: Structural design option and an increase of the surface**
The concrete wall will be raised and reinforced (if necessary). The surface will be raised such that cyclists and pedestrians are able to view the river properly as stated within the design criteria.

**Option 0: Present levee alignment; E: Double wall system**
The present concrete capping beam will be raised up to the required height. A sheet pile will placed to form a double wall system.

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Option 0: Present levee alignment; F: Dynamic system
One of the demountable systems are possible, too. Of course there are some uncertainties if these

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types of system fulfil the present and new safety requirements but are of great interest within the important area of Venlo. The dynamic systems are not considered separately as most systems have their own contractors/patents. Figure F.27 shows the dynamic system within the governing cross-section.

**Option 1: Change levee alignment to the river-side of the road; G: Structural design option and increase of the surface**

By raising the surface level height one obtain a smaller visual flood defence. Figure F.28 illustrate this levee design alternative, consisting mainly of a sheet pile. During major maintenance a concrete capping beam will be placed on top of the sheet pile. The road c.q. parking lots are separated in this alternative.

**Option 1: Change levee alignment to the river-side of the road; H: Structural design option with a glass wall system and increase of the surface at land-side**

The road and paths will be redesigned such that a glass wall system can be constructed somewhere in the middle. The new concrete part of the present levee structure will be demolished. The road/path on the river-side of alternative H can be lowered if preferred such that one experience the historical city wall more intensive. The underground infrastructural elements have to be reconstructed. A sketch

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is included in figure F.29.

**Figure F.29:** Option 1: Change levee alignment to the river-side of the road; \( H \): Structural design option with a glass wall system and increase of the surface at land-side

**Option 1: Change levee alignment to the river-side of the road; I: Structural design option and increase of the surface at land-side**

Here a sheet pile forms the structural design option with a small earthy increase of the surface at the land-side of the structure. On top of this increase a bicycle/walking and/or maintenance path can be placed. This levee design alternative is sketched inside the governing cross-section in figure F.30.

**Figure F.30:** Option 1: Change levee alignment to the river-side of the road; \( I \): Structural design option and increase of the surface at land-side

**Option 1: Change levee alignment to the river-side of the road; J: double wall system**

The execution of a multiple level building along the flood defence of section 4 will probably be planned in the upcoming few years. Designing a flood defence which can be integrated later with these buildings might be a good solution. The ground floor will primarily be used to park vehicles. Both walls of this building coincide with the sheet pile walls of this alternative. That why there is a large distance between them as can be seen at figure F.31. At both ends, a dynamic or demountable system have to be placed.
Option 1: Change levee alignment to the river-side of the road; **K**: Structural design option and double wall system
This levee design alternative is similar as alternative 4J but here the surface height increase is less to limit the visual impact of the flood defence.

Option 2: Change levee alignment to the other side of the road; **L**: Glass wall system in front of houses
An extra hall is created consisting of a concrete foundation/wall and a glass wall. Occasionally a pass-through is present which can be closed by a demountable system. Valves have to be installed within the sewage system to prevent seepage. A sketch is shown in figure F.33.

Option 2: Change levee alignment to the other side of the houses; **M**: Multiple wall system
A multiple wall system is moved as much as possible to the land-side to limit the visual impact. Here changes have to be made regarding the underground infrastructure like the sewer system to prevent leakage. A big disadvantage is the difficulty for residents to access their houses properly. Figure F.34 illustrate alternative M, section 4.
Option 2: Change levee alignment to the other side of the houses; N: Connect sheet pile with house protection system
Here a house protection system have to be constructed combined with a concrete floor and a sheet pile. The latter two structures limit seepage. At this moment it is unknown if these kind of systems will fulfil the failure mechanism "failure to close" as it consist of multiple dynamic or demountable doorways, brick-work protection elements and multiple valves. The total flood protection system have to be established up to a height of 2,6 meter.

F.3.3. Available space
The previous paragraphs defined and designed levee design alternatives and sketched them in the governing cross-sections. Sometimes an alternative does not fit within the cross-section and will not be investigated further. Table F.4 shows all alternatives per levee section and whether there is available space.

F.4. Inventory cables and pipes
With help of GIS software and available data, an indication of the present cables and pipes are shown in figure F.36. Although, there are probably more cables and pipes present, it gives some understanding about what priority types and number of pipes and cables are present.
The municipality of Venlo changed the cables and pipes along the flood defence during the execution of Q4. Since then, most of these are situated at least 5 meter from the levee, much closer to the buildings. With this knowledge one is able to screen more properly even with the little GIS-information available.
F.5. Screening alternatives

Both sections are screened with the toolbox of appendix C and shown in tables F.5 and F.6. There are many alternatives possible, especially including the variability of the road. Besides, there are some cases in where it is unclear to include a sheet pile or a concrete retaining wall in the design. The current situation includes a double road, a double walking path and parking lots at both places of the road. There are many variation like separating the road, including a separated bicycle/walking path or changing the parking lot positions. Many of these aspects are hard to screen and are strongly depending on preferences of the municipality and residents.

Two levee design alternatives of section 3 satisfy all screening criteria: B (the present demountable system) and E (glass wall system). Both alternatives will be analysed on their LCC. Furthermore, a dynamic and an alternative with a different alignment are chosen in order to check whether these alternatives are economically of interest. Therefore levee design alternatives F and L will be analysed on their LCC, too.

<table>
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<tr>
<th>Screening criteria</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
<th>G</th>
<th>H</th>
<th>I</th>
<th>J</th>
<th>K</th>
<th>L</th>
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<td>-</td>
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<td>+</td>
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<td>+</td>
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Two levee design alternatives score best during the screening procedure in section 4: B and C. Alter-
natives D, F and I are also chosen for this section to analyse further.

Table F.6: MCA ring-levee 68 Venlo, section 4

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<tr>
<th>Screening criteria</th>
<th>Alternatives</th>
<th>Stakeholders</th>
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<td>A  B  C  D  E  F  G  H  I  J  K  L  M  N</td>
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</tr>
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<td>Nature</td>
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<td>12</td>
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<tr>
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</table>
This appendix gathers all outcomes related to the investments and costs of this research. The appendix contains of two paragraphs: The basic cost estimate (G.1), which is used during the screening procedure, and the LCC estimation (G.2).

**G.1. Basic cost estimate**

A basic cost estimate has been carried out in order to screen levees on their investments. Both case studies, Arcen and Venlo, are shown here.

Construction elements and their costs per unit are present at the top right hand side. Each alternative per section has its own column where amounts are presented. Each amount is multiplied with the value per unit ans summed together. A histogram is present at the bottom part of each section.

**G.1.1. Results basic cost estimate case study Arcen**

In order to screen levee design alternatives on their investments, a basic estimation is made. Results of the investment estimate are shown in the figures below.
### BASIC COST ESTIMATE

<table>
<thead>
<tr>
<th>LCCA</th>
<th>PROJECT COSTS</th>
<th>RING-LEVEL 65: ARCEM</th>
<th>RING-LEVEL ELEMENT: 3</th>
<th>SECTION 3</th>
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<tr>
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<td>Bavalmod (surplus)</td>
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### PROJECT COSTS

| | € 3,223 | € 3,936 | € 3,805 | € 0 | € 6,690 | € 0 | € 0 | € 0 | € 0 | € 4,005 | € 6,653 | € 7,100 | € 4,443 | € 5,362 |
G.1. Basic cost estimate

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<td>Pavement</td>
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**PROJECT COSTS**

€ 2.443 € 2.051 € 3.821 € 5.578 € 7.100 € 4.093 € 4.014

MSc thesis Bart Broers
G.1.2. Results basic cost estimate case study Venlo

In order to screen levee design alternatives on their investments, a basic estimation is made. Results of the investment estimate are shown in the figures below.

## BASIC COST ESTIMATE

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<th>Units</th>
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<td>130/m³</td>
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<td>Roads</td>
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<tr>
<td>Asphalt</td>
<td>110/m³</td>
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<tr>
<td>Pavement</td>
<td>81/m³</td>
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<tr>
<td>Maintenance path</td>
<td>31/m³</td>
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</tr>
<tr>
<td>Soil</td>
<td></td>
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<tr>
<td>Sand + transport</td>
<td>12/m³</td>
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<tr>
<td>Clay + transport (E.B.1)</td>
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<tr>
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<td>18/m³</td>
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<tr>
<td>Executing</td>
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<tr>
<td>Cover layer + executing (0.30 m)</td>
<td>8/m³</td>
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## PROJECT COSTS

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<th>Value (€)</th>
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<tr>
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<td>2.10</td>
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<td>2.01</td>
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</tr>
<tr>
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</tbody>
</table>

MSc thesis Bart Broers
G.2. LCCA

The analysis of the LCC consists of a general part, where prices per aspect are formed based upon GWKosten.nl, and a second part wherein the LCC are calculated per alternative. These prices are not yet discounted to their present value. Whether or not an aspect depend on the variables within the optimization procedure is stated at the right hand side. A C means constant while a H1 or H2 means a dependence, related to $\Delta h_1$ and $\Delta h_2$ respectively. All aspecty depending on these parameters are excluded from the totals. After discounting, these totals are combined in one constant investment ($I_{c.0}$ and $I_{c.m}$) in order to include them in the Matlab script. A summery of the resulted Matlab investments, benefits and NPV’s are given per case study as well.

---

MSc thesis Bart Broers
## Demolition

<table>
<thead>
<tr>
<th>Description</th>
<th>Unit</th>
<th>Time per</th>
<th>Price per Unit</th>
<th>Unit Cost</th>
</tr>
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<tbody>
<tr>
<td>Demolition masonry</td>
<td></td>
<td></td>
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<tr>
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<td>m2/h</td>
<td>0.06 h/m2</td>
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<tr>
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<td>m2/h</td>
<td>0.13 h/m2</td>
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<td>€ 10.50</td>
</tr>
<tr>
<td>Truck</td>
<td>m2/h</td>
<td>0.13 h/m2</td>
<td>84.00</td>
<td>€ 10.50</td>
</tr>
<tr>
<td>Deposit costs</td>
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<td></td>
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<tr>
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<td>€ 17,69</td>
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<tr>
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<td>Demolition concrete floor</td>
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<td></td>
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</tr>
<tr>
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<td>Demolish anchors</td>
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<td>Cutting and braking anchors</td>
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<td></td>
<td></td>
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<td>Pulling sheet pile</td>
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<tr>
<td>Loading/unloading</td>
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<td></td>
<td></td>
<td>€ 8.40</td>
</tr>
<tr>
<td>Truck</td>
<td></td>
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<td>€ 8.40</td>
</tr>
<tr>
<td>Recycling steel</td>
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<td></td>
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<tr>
<td>Demolish layered glass wall</td>
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<tr>
<td>Layered glass wall</td>
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<td></td>
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<td>Steel frame</td>
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<td>1,25 kg/m1</td>
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<td>Rubber</td>
<td>m1/m2</td>
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<td></td>
<td>€ 0.60</td>
</tr>
<tr>
<td>Stee feet, 0,20x0,25x0,02</td>
<td></td>
<td></td>
<td></td>
<td>€ 0.00</td>
</tr>
<tr>
<td>Truck with crane</td>
<td>m1/h</td>
<td>0.08</td>
<td>76.00</td>
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<tr>
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<td>m1/h</td>
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<td>73.10</td>
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<td>Transport</td>
<td>m1/h</td>
<td>0.04</td>
<td>76.00</td>
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<td>m</td>
<td>1.00</td>
<td>73.10</td>
<td>€ 3.66</td>
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<td>76.00</td>
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<td>-5.25</td>
<td>-</td>
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<tr>
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<td>m</td>
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<td>36.55</td>
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<td>Storage</td>
<td>st</td>
<td></td>
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<td>€ 20.00</td>
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<td>Replacing tree</td>
<td>st</td>
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<td>76.00</td>
<td>€ 7.60</td>
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<td>Excavating present situation</td>
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<td>36.55</td>
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<td>Filling</td>
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<td>m3/u</td>
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<td>76.00</td>
<td>€ 2.53</td>
</tr>
<tr>
<td>Excavation worker</td>
<td>m3/u</td>
<td>0.03</td>
<td>36.55</td>
<td>€ 1.22</td>
</tr>
<tr>
<td>vibroplate/vibropounder</td>
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<td></td>
<td></td>
<td>€ 0.17</td>
</tr>
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<td>Ground deposit to depot</td>
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<td></td>
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<td>Truck</td>
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</tr>
<tr>
<td>Loading</td>
<td>h</td>
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<tr>
<td>Speed</td>
<td>km/h</td>
<td>30.00</td>
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<tr>
<td>Transport away</td>
<td>h</td>
<td>0.13</td>
<td>€ 8.00</td>
<td></td>
</tr>
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</table>
## Unloading

- **Time:** 0.08 h
- **Cost:** € 5.00

## Transport back

- **Time:** 0.13 h
- **Cost:** € 8.00

## Netto Cyclustime

- **Time:** 0.58 h
- **Cost:** € 34.50

## Loading truck

- **Time:** 3.00 h

## Waittime

- **Time:** 0.10 h
- **Cost:** € 6.00

## Brutto cyclustime

- **Time:** 0.68 h
- **Cost:** € 40.50

## Production

- **Time:** 26.67 m3/h
- **Number of trucks:** 3.00

## Production per hour

- **Total:** 84.00 €
- **Cost per hour:** € 3.15

### Filling and supply clay

<table>
<thead>
<tr>
<th>Description</th>
<th>Unit</th>
<th>Time per</th>
<th>Price per unit</th>
<th>Cost per unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hydraulic excavator 1.2 m3</td>
<td>30.00 m3/u</td>
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<td>€ 2.53 m3/u</td>
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<tr>
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<td>€ 36.00</td>
<td>€ 1.20 m3/u</td>
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<tr>
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<td>1.00 m3</td>
<td>2.00 km</td>
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<td>€ 6.00</td>
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<tr>
<td>Processing of clay</td>
<td>1.00 m3</td>
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<td>€ 1.59</td>
<td>€ 1.59</td>
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</table>

### Concrete and mesonry

#### Concrete floor

<table>
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<th>Price per unit</th>
<th>Cost per unit</th>
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</thead>
<tbody>
<tr>
<td>Transport + poring</td>
<td>1.00 m3</td>
<td></td>
<td>€ 125.00</td>
<td>€ 125.00 m3</td>
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<tr>
<td>Formwork / falsework</td>
<td>1.00 m3/m3</td>
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<td>€ 25.40 m3/m3</td>
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<tr>
<td>Reinforcement steel</td>
<td>125.00 kg/m3</td>
<td>1.00 m3</td>
<td>€ 1.49</td>
<td>€ 185.94 m3</td>
</tr>
</tbody>
</table>

#### Concrete wall

<table>
<thead>
<tr>
<th>Description</th>
<th>Unit</th>
<th>Time per</th>
<th>Price per unit</th>
<th>Cost per unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transport + poring</td>
<td>1.00 m3</td>
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<td>€ 140.00</td>
<td>€ 140.00 m3</td>
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<td>1.00 m3</td>
<td>€ 1.69</td>
<td>€ 210.94 m3</td>
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#### Concrete capping beam

<table>
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<tr>
<th>Description</th>
<th>Unit</th>
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<th>Price per unit</th>
<th>Cost per unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transport + poring</td>
<td>1.00 m3</td>
<td></td>
<td>€ 140.00</td>
<td>€ 140.00 m3</td>
</tr>
<tr>
<td>Formwork / falsework</td>
<td>10.00 m2/m3</td>
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<td>€ 80.35</td>
<td>€ 803.50 m2/m3</td>
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<td>Reinforcement steel 16 mm</td>
<td>125.00 kg/m3</td>
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<td>€ 1.49</td>
<td>€ 185.94 m3</td>
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#### Mesonry

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<th>Cost per unit</th>
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</thead>
<tbody>
<tr>
<td>Brick + mortal</td>
<td>95.00 p/m2</td>
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<td>€ 95.00 p/m2</td>
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<tr>
<td>Mason</td>
<td>1.25 m2/h</td>
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#### Dilatation joint

<table>
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<th>Cost per unit</th>
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</thead>
<tbody>
<tr>
<td>labour</td>
<td>12.00 m/h</td>
<td>0.08 h/m2</td>
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<td>€ 3.05/m</td>
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<td>Poring joint</td>
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<td>6.00 m</td>
<td>€ 15.00</td>
<td>€ 2.50/m</td>
</tr>
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<td>Joint material</td>
<td>1.00 m</td>
<td>6.00 m</td>
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<td>€ 1.33/m</td>
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</table>

#### Connecting with existing wall

<table>
<thead>
<tr>
<th>Description</th>
<th>Unit</th>
<th>Time per</th>
<th>Price per unit</th>
<th>Cost per unit</th>
</tr>
</thead>
<tbody>
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<td>labour</td>
<td>4.00 m/h</td>
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<td>€ 9.14/m</td>
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<td>€ 3.75/m</td>
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<td>1.61 kg/m</td>
<td>€ 1.25</td>
<td>€ 2.01/m</td>
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### Sheet pile and anchors

#### Installation sheet pile

<table>
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<th>Description</th>
<th>Unit</th>
<th>Time per</th>
<th>Price per unit</th>
<th>Cost per unit</th>
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</thead>
<tbody>
<tr>
<td>Supply steel sheetpile (AZ26)</td>
<td>1.00 m long</td>
<td>0.16 m/m2</td>
<td>€ 950.00</td>
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<tr>
<td>heier</td>
<td>27.00 m2/h</td>
<td>0.04 h</td>
<td>€ 36.00</td>
<td>€ 1.33/m</td>
</tr>
<tr>
<td>labout to set out</td>
<td>50.00 m/h</td>
<td>0.02 h</td>
<td>€ 34.00</td>
<td>€ 0.68/m</td>
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<td>Piling equipment</td>
<td>27.00 m2/h</td>
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<td>plasma cutting</td>
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<td>Joining with concrete</td>
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<td>€ 100.00/m</td>
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#### Installation anchors

<table>
<thead>
<tr>
<th>Description</th>
<th>Unit</th>
<th>Time per</th>
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<th>Cost per unit</th>
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</thead>
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<td>Center to center</td>
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<td>10.00 m</td>
<td>0.36 p/m1</td>
<td>€ 50.00</td>
<td>€ 178.57/m</td>
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<tr>
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<td>5.00 p/h</td>
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<td>€ 120.00</td>
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<td>0.36 p/m1</td>
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<td>€ 178.57/m</td>
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<td>0.07 h</td>
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<td>Description</td>
<td>Unit</td>
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<td>Price per</td>
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<td>----------</td>
<td>-----------</td>
</tr>
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<td>1.00 m</td>
<td>65.00 €</td>
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<tr>
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<td>8.00 m/uur</td>
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<td>Rubber</td>
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<td>1 every 24 m (1 per two houses)</td>
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<td>Supply stairs</td>
<td>5.00 m</td>
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<td>Mowing</td>
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<td>Tracktor with clapper</td>
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<td>0.0016 h/m²</td>
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</tr>
<tr>
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<td>0.0001 hec</td>
<td>4.500,00 €</td>
<td>€ 0.45/m²</td>
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<td>price per unit</td>
<td>€ 3.60/m²</td>
</tr>
<tr>
<td>Transport</td>
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<td>0.003 h</td>
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<td>Cleaning</td>
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<td>0.0400 h</td>
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<td>Hydraulic excavator</td>
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<td>€ 2.53/m²</td>
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<tr>
<td>Labour 2p</td>
<td>30.00 m³/u</td>
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<td>€ 2.44/m²</td>
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<tr>
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<td>price per unit</td>
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<td>4000 m</td>
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<td>€ 18,75/m</td>
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<td>price per unit</td>
<td>€ 18.75/m</td>
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<td>1</td>
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<tr>
<td>price during closure</td>
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<td>price per unit</td>
<td>€ 2.19/m</td>
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<td>20 m</td>
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<td>Starting closure</td>
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<td>100 €</td>
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<tr>
<td>number of closures per 25 year</td>
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<tr>
<td>price during closure</td>
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<td>Analysis time</td>
<td>100 year</td>
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### Project costs

<table>
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<tr>
<th>Description</th>
<th>Amount</th>
<th>Unit</th>
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<th>Total Price per m²</th>
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<tbody>
<tr>
<td>Land site preparation</td>
<td>15.00</td>
<td>m²</td>
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<td>€ 45.00</td>
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<td>remove non water retaining objects</td>
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<td>€ 1.00</td>
<td>€ 8.00</td>
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<td>€ 11.25</td>
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<td>1.00</td>
<td>m</td>
<td>€ 128.68</td>
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<td>13.50</td>
<td>m</td>
<td>€ 155.06</td>
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<td>€ 451.78</td>
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<td>0.45</td>
<td>m³</td>
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<td>€ 323.63</td>
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<td>m³</td>
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<td>1.00</td>
<td>m</td>
<td>€ 277.46</td>
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<td>m³</td>
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<td>€ 92.70</td>
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<td>25.00</td>
<td>m²</td>
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<td>Safety and usage objects (stairs)</td>
<td>2.00</td>
<td>m</td>
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<td>€ 25.31</td>
<td>€ 25.31</td>
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<td>€ 21.84</td>
<td>€ 21.84</td>
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### Service costs (per year)

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<td>Monthly inspection</td>
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<td>/m/J</td>
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<td>Mowing grass</td>
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### Service costs (per 10 years)

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<tbody>
<tr>
<td>Seeding once every 10 year</td>
<td>3.00</td>
<td>m²</td>
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### Reactive maintenance (per year)

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<tr>
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### Major Maintenance (per 50 years)

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<td>4.80</td>
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<tr>
<td>remove non water retaining objects</td>
<td>4.80</td>
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<td>Installation reinforced concrete capping beam</td>
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<td>m³</td>
<td>€ 1.129.44</td>
<td>€ 451.78</td>
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<td>0.60</td>
<td>m³</td>
<td>€ 719.19</td>
<td>€ 431.51</td>
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<td>fill two wall system</td>
<td>4.80</td>
<td>m³</td>
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<td>Finishing and seeding ground</td>
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<td>Take away and store maintenance path</td>
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<td>Installation of maintenance path</td>
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### End-of-Life Costs

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<td>€ 17.69</td>
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<tr>
<td>Demolisch concrete capping beam</td>
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<td>€ 17.69</td>
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<tr>
<td>Demolisch anchors</td>
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### Section 1: Alternative D

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<td>€ 8,00</td>
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</tr>
<tr>
<td>soil excavating</td>
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<td>€ 1.693,63</td>
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<td>0,40</td>
<td>m3</td>
<td>€ 1.129,44</td>
<td>€ 451,78</td>
<td>C</td>
</tr>
<tr>
<td>Installation reinforced concrete wall</td>
<td>0,45</td>
<td>m3</td>
<td>€ 719,19</td>
<td>€ 323,63</td>
<td>C</td>
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<tr>
<td>Installation reinforced concrete wall</td>
<td>0,30</td>
<td>m3</td>
<td>€ 719,19</td>
<td>€ 215,76</td>
<td>C</td>
</tr>
<tr>
<td>Installation anchors</td>
<td>1,00</td>
<td>m</td>
<td>€ 277,46</td>
<td>€ 277,46</td>
<td>C</td>
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<tr>
<td>Installation of expansion joints</td>
<td>1,00</td>
<td>m</td>
<td>€ 6,88</td>
<td>€ 6,88</td>
<td>C</td>
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<tr>
<td>restore soil</td>
<td>16,50</td>
<td>m3</td>
<td>€ 1,39</td>
<td>€ 22,85</td>
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<td>Gain soil from other side of the river</td>
<td>16,80</td>
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<td>€ 7,07</td>
<td>€ 118,75</td>
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<td>fill two wall system</td>
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<td>€ 47,02</td>
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<td>Clay supply and filling</td>
<td>4,80</td>
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<td>25,00</td>
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<td>Safety and usage objects</td>
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<td>€ 12,08</td>
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<tr>
<td>Maintenance path</td>
<td>1,00</td>
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<td>€ 21,84</td>
<td>€ 21,84</td>
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#### Service costs (per year)

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<tbody>
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<td>€ 1,50</td>
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<td>Mowing grass</td>
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#### Service costs (per 10 years)

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</thead>
<tbody>
<tr>
<td>Seeding once every 10 year</td>
<td>2,50</td>
<td>m2</td>
<td>€ 0,45</td>
<td>€ 1,13</td>
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#### Reactive maintenance (per year)

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<td></td>
<td></td>
<td>€ 0,00</td>
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#### Major Maintenance (per 50 years)

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<tbody>
<tr>
<td>Land site preparation</td>
<td>4,80</td>
<td>m2</td>
<td>€ 3,00</td>
<td>€ 14,40</td>
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<td>remove non water retaining objects</td>
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<td>m2</td>
<td>€ 1,00</td>
<td>€ 4,80</td>
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<td>0,10</td>
<td>m3</td>
<td>€ 1,129,44</td>
<td>€ 112,94</td>
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<td>Installation reinforced concrete wall</td>
<td>0,30</td>
<td>m3</td>
<td>€ 719,19</td>
<td>€ 215,76</td>
<td>H2</td>
</tr>
<tr>
<td>Finishing and seeding ground</td>
<td>4,80</td>
<td>m2</td>
<td>€ 2,00</td>
<td>€ 9,60</td>
<td>C</td>
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<td><strong>Subtotal execution costs</strong></td>
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<td><strong>Direct execution costs</strong></td>
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<tr>
<td>Execution costs</td>
<td>7,00%</td>
<td></td>
<td></td>
<td>€ 10,91</td>
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<tr>
<td>Profit and risks</td>
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#### End-of-Life Costs

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<tbody>
<tr>
<td>Demolisch concrete capping beam</td>
<td>1,00</td>
<td>m</td>
<td>€ 17,69</td>
<td>€ 17,69</td>
<td>C</td>
</tr>
<tr>
<td>Demolisch concrete capping beam</td>
<td>1,00</td>
<td>m</td>
<td>€ 17,69</td>
<td>€ 17,69</td>
<td>C</td>
</tr>
<tr>
<td>Demolisch anchors</td>
<td>1,00</td>
<td>m</td>
<td>€ 22,32</td>
<td>€ 22,32</td>
<td>C</td>
</tr>
<tr>
<td>soil excavation</td>
<td>16,50</td>
<td>m3</td>
<td>€ 3,73</td>
<td>€ 61,60</td>
<td>C</td>
</tr>
<tr>
<td>Remove sheet piles + recycling</td>
<td>15,00</td>
<td>m2</td>
<td>€ 21,30</td>
<td>€ 319,51</td>
<td>C</td>
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<tr>
<td>Remove maintenance path</td>
<td>3,00</td>
<td>m2</td>
<td>€ 10,00</td>
<td>€ 30,00</td>
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<td><strong>Total</strong></td>
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<td></td>
<td></td>
<td>€ 468,82</td>
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**SECTION 1:**

**Alternative K:**  

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<th>Project costs</th>
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<th>unit</th>
<th>price/unit</th>
<th>price per m1</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Land site preparation</td>
<td>15,00</td>
<td>m²</td>
<td>€ 3,00</td>
<td>€ 45,00</td>
<td>C</td>
</tr>
<tr>
<td>remove non water retaining objects</td>
<td>8,00</td>
<td>m²</td>
<td>€ 1,00</td>
<td>€ 8,00</td>
<td>C</td>
</tr>
<tr>
<td>soil excavating for foundation</td>
<td>3,00</td>
<td>m³</td>
<td>€ 3,75</td>
<td>€ 11,26</td>
<td>C</td>
</tr>
<tr>
<td>remove partly present levee structure (masonry + concrete?)</td>
<td>2,50</td>
<td>m²</td>
<td>€ 18,88</td>
<td>€ 47,20</td>
<td>C</td>
</tr>
<tr>
<td>Installation of sheet pile AZ-26</td>
<td>1,00</td>
<td>m</td>
<td>€ 128,68</td>
<td>€ 128,68</td>
<td>C</td>
</tr>
<tr>
<td>Installation of sheet pile AZ-26</td>
<td>9,00</td>
<td>m</td>
<td>€ 155,06</td>
<td>€ 1,395,58</td>
<td>C</td>
</tr>
<tr>
<td>Installation reinforced concrete floor</td>
<td>1,25</td>
<td>m³</td>
<td>€ 371,25</td>
<td>€ 464,06</td>
<td>C</td>
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<tr>
<td>Installation reinforced concrete capping beam</td>
<td>0,40</td>
<td>m³</td>
<td>€ 1,096,25</td>
<td>€ 438,50</td>
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<tr>
<td>Installation anchors</td>
<td>1,00</td>
<td>m</td>
<td>€ 271,64</td>
<td>€ 271,64</td>
<td>C</td>
</tr>
<tr>
<td>Installation of expansion joints</td>
<td>1,00</td>
<td>m²</td>
<td>€ 26,00</td>
<td>€ 26,00</td>
<td>C</td>
</tr>
<tr>
<td>Installation of glass wall</td>
<td>1,00</td>
<td>m²</td>
<td>€ 402,00</td>
<td>€ 402,00</td>
<td>H1</td>
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<tr>
<td>filling soil</td>
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<td>m³</td>
<td>€ 7,67</td>
<td>€ 103,55</td>
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<tr>
<td>Gain soil from other side of the river</td>
<td>13,50</td>
<td>m³</td>
<td>€ 7,67</td>
<td>€ 103,55</td>
<td>C</td>
</tr>
<tr>
<td>Filling clay</td>
<td>7,50</td>
<td>m³</td>
<td>€ 19,31</td>
<td>€ 144,85</td>
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<tr>
<td>Finishing and seeding ground</td>
<td>25,00</td>
<td>m²</td>
<td>€ 2,00</td>
<td>€ 50,00</td>
<td>C</td>
</tr>
<tr>
<td>Safety and usage objects</td>
<td>1,00</td>
<td>m</td>
<td>€ 60,42</td>
<td>€ 60,42</td>
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<tr>
<td>Maintenance path</td>
<td>1,00</td>
<td>m</td>
<td>€ 21,84</td>
<td>€ 21,84</td>
<td>C</td>
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</table>

**Subtotal execution costs**  
€ 3,884,51  
Further detailing 10,00%  
€ 388,45  
**Direct execution costs**  
€ 4,272,96  
Execution costs 7,00%  
€ 299,11  
Profit and risks 5,00%  
€ 213,65  
Total  
€ 4,785,72

**Service costs (per year)**

<table>
<thead>
<tr>
<th>Service costs (per year)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Monthly inspection</td>
<td>1,00 /m/J</td>
</tr>
<tr>
<td>Mowing grass</td>
<td>3,00 m²</td>
</tr>
<tr>
<td>Glass cleaning at both sides per year</td>
<td>1,00 m²</td>
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<tr>
<td><strong>Total</strong></td>
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</table>

**Service costs (per 10 years)**

<table>
<thead>
<tr>
<th>Service costs (per 10 years)</th>
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</thead>
<tbody>
<tr>
<td>Seeding once every 10 year</td>
<td>3,00 m²</td>
</tr>
<tr>
<td>Reinstallation of rubbers and kit</td>
<td>1,00 m</td>
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<tr>
<td><strong>Total</strong></td>
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**Reactive maintenance (per year)**

<table>
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</thead>
<tbody>
<tr>
<td>Broken glass (1m² every 200 m² per year)</td>
<td>0,01 m²</td>
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<tr>
<td><strong>Total</strong></td>
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**Major Maintenance (per 50 years)**

<table>
<thead>
<tr>
<th>Major Maintenance (per 50 years)</th>
<th></th>
</tr>
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<tbody>
<tr>
<td>Remove existing glass wall</td>
<td>1,00 m</td>
</tr>
<tr>
<td>Installation of glass wall dependence of height</td>
<td>1,00 m²</td>
</tr>
<tr>
<td>Installation of glass wall dependence of height</td>
<td>1,00 m²</td>
</tr>
<tr>
<td><strong>Subtotal execution costs</strong></td>
<td></td>
</tr>
<tr>
<td>Further detailing 10,00%</td>
<td></td>
</tr>
<tr>
<td><strong>Direct execution costs</strong></td>
<td></td>
</tr>
<tr>
<td>Execution costs 7,00%</td>
<td></td>
</tr>
<tr>
<td>Profit and risks 5,00%</td>
<td></td>
</tr>
<tr>
<td><strong>Total</strong></td>
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**End-of-Life Costs**

<table>
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<tr>
<th>End-of-Life Costs</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Remove existing glass wall</td>
<td>1,00 m</td>
</tr>
<tr>
<td>Demolisch concrete capping beam</td>
<td>1,00 m</td>
</tr>
<tr>
<td>Demolisch concrete floor</td>
<td>1,25 m³</td>
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<tr>
<td>soil excavation</td>
<td>4,00 m³</td>
</tr>
<tr>
<td>Demolisch anchors</td>
<td>1,00 m</td>
</tr>
<tr>
<td>Remove sheet piles + recycling</td>
<td>9,00 m²</td>
</tr>
<tr>
<td>Remove maintenance path</td>
<td>3,00 m²</td>
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### SECTION 2: Alternative C:

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<tbody>
<tr>
<td>Land site preparation</td>
<td>15,00</td>
<td>m²</td>
<td>€ 3,00</td>
<td>€ 45,00</td>
</tr>
<tr>
<td>remove non water retaining objects</td>
<td>8,00</td>
<td>m²</td>
<td>€ 1,00</td>
<td>€ 8,00</td>
</tr>
<tr>
<td>soil excavating for foundation</td>
<td>3,00</td>
<td>m³</td>
<td>€ 3,75</td>
<td>€ 11,26</td>
</tr>
<tr>
<td>Installation of sheet pile AZ-26</td>
<td>1,00</td>
<td>m</td>
<td>€ 128,68</td>
<td>€ 128,68</td>
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<td>Installation of sheet pile AZ-26</td>
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<td>m²</td>
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<td>€ 2,015,84</td>
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<td>m²</td>
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<td>Installation reinforced concrete capping beam</td>
<td>0,40</td>
<td>m³</td>
<td>€ 1,129,44</td>
<td>€ 451,78</td>
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<tr>
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<td>0,45</td>
<td>m³</td>
<td>€ 719,19</td>
<td>€ 323,63</td>
</tr>
<tr>
<td>Installation reinforced concrete wall</td>
<td>0,30</td>
<td>m³</td>
<td>€ 719,19</td>
<td>€ 215,76</td>
</tr>
<tr>
<td>Installation of expansion joints</td>
<td>1,00</td>
<td>m</td>
<td>€ 6,88</td>
<td>€ 6,88</td>
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<tr>
<td>Installation of expansion joints</td>
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<td>m</td>
<td>€ 6,88</td>
<td>€ 6,88</td>
</tr>
<tr>
<td>Installation reinforced concrete capping beam</td>
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<td>€ 719,19</td>
<td>€ 431,51</td>
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<td>m</td>
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<td>€ 22,32</td>
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<tr>
<td>Installation reinforced concrete capping beam</td>
<td>0,40</td>
<td>m³</td>
<td>€ 1,129,44</td>
<td>€ 451,78</td>
</tr>
<tr>
<td>Installation reinforced concrete capping beam</td>
<td>0,60</td>
<td>m³</td>
<td>€ 719,19</td>
<td>€ 431,51</td>
</tr>
<tr>
<td>Installation of expansion joints</td>
<td>1,00</td>
<td>m</td>
<td>€ 6,88</td>
<td>€ 6,88</td>
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<tr>
<td>Maintenance path</td>
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<td>Total</td>
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</tbody>
</table>

### Service costs (per year)

<table>
<thead>
<tr>
<th>Service costs (per year)</th>
<th>amount</th>
<th>unit</th>
<th>price/unit</th>
<th>price per m1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monthly inspection</td>
<td>1,00</td>
<td>/m/J</td>
<td>€ 1,50</td>
<td>€ 1,50</td>
</tr>
<tr>
<td>Mowing grass</td>
<td>4,80</td>
<td>m²</td>
<td>€ 0,39</td>
<td>€ 1,87</td>
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<tr>
<td>Total</td>
<td>€ 3,37</td>
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### Service costs (per 10 years)

<table>
<thead>
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<th>Service costs (per 10 years)</th>
<th>amount</th>
<th>unit</th>
<th>price/unit</th>
<th>price per m1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seeding once every 10 year</td>
<td>4,80</td>
<td>m²</td>
<td>€ 0,45</td>
<td>€ 2,16</td>
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<td>Total</td>
<td>€ 2,16</td>
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</table>

### Reactive maintenance (per year)

<table>
<thead>
<tr>
<th>Reactive maintenance (per year)</th>
<th>amount</th>
<th>unit</th>
<th>price/unit</th>
<th>price per m1</th>
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</thead>
<tbody>
<tr>
<td>Total</td>
<td>€ 0,00</td>
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</table>

### Major Maintenance (per 50 years)

<table>
<thead>
<tr>
<th>Major Maintenance (per 50 years)</th>
<th>amount</th>
<th>unit</th>
<th>price/unit</th>
<th>price per m1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Land site preparation</td>
<td>4,80</td>
<td>m²</td>
<td>€ 3,00</td>
<td>€ 14,40</td>
</tr>
<tr>
<td>remove non water retaining objects</td>
<td>4,80</td>
<td>m²</td>
<td>€ 1,00</td>
<td>€ 4,80</td>
</tr>
<tr>
<td>Installation reinforced concrete capping beam</td>
<td>0,40</td>
<td>m³</td>
<td>€ 1,129,44</td>
<td>€ 451,78</td>
</tr>
<tr>
<td>Installation reinforced concrete capping beam</td>
<td>0,60</td>
<td>m³</td>
<td>€ 719,19</td>
<td>€ 431,51</td>
</tr>
<tr>
<td>Installation of expansion joints</td>
<td>1,00</td>
<td>m</td>
<td>€ 6,88</td>
<td>€ 6,88</td>
</tr>
<tr>
<td>fill two wall system</td>
<td>4,80</td>
<td>m³</td>
<td>€ 10,82</td>
<td>€ 51,94</td>
</tr>
<tr>
<td>Finishing and seeding ground</td>
<td>15,00</td>
<td>m²</td>
<td>€ 2,00</td>
<td>€ 30,00</td>
</tr>
<tr>
<td>Safety and usage objects (stairs)</td>
<td>4,00</td>
<td>m</td>
<td>€ 25,31</td>
<td>€ 101,25</td>
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<tr>
<td>Safety and usage objects (stairs)</td>
<td>1,00</td>
<td>m</td>
<td>€ 25,31</td>
<td>€ 25,31</td>
</tr>
<tr>
<td>Maintenance path</td>
<td>1,00</td>
<td>m</td>
<td>€ 21,84</td>
<td>€ 21,84</td>
</tr>
<tr>
<td>Subtotal execution costs</td>
<td>€ 531,80</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Further detailing</td>
<td>10,00%</td>
<td>€ 53,18</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Direct execution costs</td>
<td>€ 584,98</td>
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<td></td>
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<tr>
<td>Execution costs</td>
<td>7,00%</td>
<td>€ 40,95</td>
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<tr>
<td>Profit and risks</td>
<td>5,00%</td>
<td>€ 29,25</td>
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<tr>
<td>Total</td>
<td>€ 655,18</td>
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### End-of-Life Costs

<table>
<thead>
<tr>
<th>End-of-Life Costs</th>
<th>amount</th>
<th>unit</th>
<th>price/unit</th>
<th>price per m1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Demolisch concrete capping beam</td>
<td>1,00</td>
<td>m</td>
<td>€ 17,69</td>
<td>€ 17,69</td>
</tr>
<tr>
<td>Demolisch concrete capping beam</td>
<td>1,00</td>
<td>m</td>
<td>€ 17,69</td>
<td>€ 17,69</td>
</tr>
<tr>
<td>Demolisch anchors</td>
<td>1,00</td>
<td>m</td>
<td>€ 22,32</td>
<td>€ 22,32</td>
</tr>
<tr>
<td>soil excavation</td>
<td>12,00</td>
<td>m³</td>
<td>€ 3,73</td>
<td>€ 44,80</td>
</tr>
<tr>
<td>Remove sheet piles + recycling</td>
<td>15,00</td>
<td>m²</td>
<td>€ 21,30</td>
<td>€ 319,51</td>
</tr>
<tr>
<td>Remove maintenance path</td>
<td>3,00</td>
<td>m²</td>
<td>€ 10,00</td>
<td>€ 30,00</td>
</tr>
<tr>
<td>Total</td>
<td>€ 452,02</td>
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</table>
SECTION 2:

**Alternative B:**

<table>
<thead>
<tr>
<th>Project costs</th>
<th>amount</th>
<th>unit</th>
<th>price/unit</th>
<th>price per m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>Land site preparation</td>
<td>25,00</td>
<td>m²</td>
<td>€ 3.00</td>
<td>€ 75,00</td>
</tr>
<tr>
<td>remove non water retaining objects</td>
<td>12,00</td>
<td>m²</td>
<td>€ 1.00</td>
<td>€ 12,00</td>
</tr>
<tr>
<td>Demolition existing masonry walls</td>
<td>2,00</td>
<td>m²</td>
<td>€ 18.91</td>
<td>€ 37,83</td>
</tr>
<tr>
<td>Gain soil from other side of the river</td>
<td>27,10</td>
<td>m³</td>
<td>€ 7.07</td>
<td>€ 191,55</td>
</tr>
<tr>
<td>Gain soil from other side of the river</td>
<td>38,70</td>
<td>m³</td>
<td>€ 7.07</td>
<td>€ 273,54</td>
</tr>
<tr>
<td>fill soil</td>
<td>27,10</td>
<td>m³</td>
<td>€ 3.92</td>
<td>€ 106,19</td>
</tr>
<tr>
<td>fill soil</td>
<td>38,70</td>
<td>m³</td>
<td>€ 3.92</td>
<td>€ 151,64</td>
</tr>
<tr>
<td>Clay supply and filling</td>
<td>24,20</td>
<td>m³</td>
<td>€ 19.31</td>
<td>€ 467,38</td>
</tr>
<tr>
<td>Finishing and seeding ground</td>
<td>30,00</td>
<td>m²</td>
<td>€ 2.00</td>
<td>€ 60,00</td>
</tr>
<tr>
<td>Maintenance path</td>
<td>1,00</td>
<td>m</td>
<td>€ 21.84</td>
<td>€ 21,84</td>
</tr>
</tbody>
</table>

**Subtotal execution costs**

€ 1.375,13

**Direct execution costs**

€ 1.512,65

**Execution costs**

€ 105,89

**Profit and risks**

€ 75,63

**Total**

€ 1.694,17

**Service costs (per year)**

- Monthly inspection: 1,00 /m/J € 1.50 € 1.50
- Mowing grass: 25,00 m² € 0.39 € 9.75

**Total**

€ 11.25

**Service costs (per 10 years)**

- Seeding once every 10 year: 25,00 m² € 0.45 € 11.25

**Total**

€ 11.25

**Reactive maintenance (per year)**

**Total**

€ 0.00

**Major Maintenance (per 50 years)**

- Land site preparation: 25,00 m² € 3.00 € 75,00
- Excavating cover-layer: 11,61 m³ € 3.75 € 43,56
- Clay supply and filling: 38,70 m³/m € 19.31 € 747,43
- Reposition cover-layer: 11,61 m³ € 3.92 € 45,49
- Finishing and seeding ground: 25,00 m² € 2.00 € 50,00
- Take away and store maintenance path: 3,00 m² € 10.00 € 30,00

**Installation of maintenance path**

- 1,00 m € 21.84 € 21,84

**Subtotal execution costs**

€ 270,89

**Direct execution costs**

€ 297,98

**Execution costs**

€ 20,86

**Profit and risks**

€ 14,90

**Total**

€ 333,73

**End-of-Life Costs**

- soil excavation: 82,25 € 3.75 € 308,57
- Remove maintenance path: 3,00 m² € 10.00 € 30,00

**Total**

€ 338,57
### Project Costs

<table>
<thead>
<tr>
<th>Description</th>
<th>Amount</th>
<th>Unit</th>
<th>Price/Unit</th>
<th>Price per M1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Land site preparation</td>
<td>15.00</td>
<td>m²</td>
<td>€ 3,00</td>
<td>€ 45,00</td>
</tr>
<tr>
<td>Remove non water retaining objects</td>
<td>8.00</td>
<td>m²</td>
<td>€ 1,00</td>
<td>€ 8,00</td>
</tr>
<tr>
<td>Soil excavating for foundation</td>
<td>3.00</td>
<td>m³</td>
<td>€ 3,75</td>
<td>€ 11,26</td>
</tr>
<tr>
<td>Installation of sheet pile AZ-26</td>
<td>1.00</td>
<td>m</td>
<td>€ 128,68</td>
<td>€ 128,68</td>
</tr>
<tr>
<td>Installation of sheet pile AZ-26</td>
<td>6.00</td>
<td>m</td>
<td>€ 155,06</td>
<td>€ 930,39</td>
</tr>
<tr>
<td>Installation reinforced concrete floor</td>
<td>1.25</td>
<td>m³</td>
<td>€ 100,00</td>
<td>€ 125,00</td>
</tr>
<tr>
<td>Installation reinforced concrete capping beam</td>
<td>0.40</td>
<td>m³</td>
<td>€ 1,129.44</td>
<td>€ 451.78</td>
</tr>
<tr>
<td>Installation reinforced concrete wall</td>
<td>0.30</td>
<td>m³</td>
<td>€ 719.19</td>
<td>€ 215.76</td>
</tr>
<tr>
<td>Installation reinforced concrete wall</td>
<td>0.30</td>
<td>m³</td>
<td>€ 719.19</td>
<td>€ 215.76</td>
</tr>
<tr>
<td>Installation of expansion joints</td>
<td>1.00</td>
<td>m</td>
<td>€ 6.88</td>
<td>€ 6.88</td>
</tr>
<tr>
<td>Restore soil</td>
<td>16.50</td>
<td>m³</td>
<td>€ 1.39</td>
<td>€ 22.85</td>
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<tr>
<td>Gain soil from other side of the river</td>
<td>10.60</td>
<td>m³</td>
<td>€ 7.07</td>
<td>€ 74.92</td>
</tr>
<tr>
<td>Fill two wall system</td>
<td>3.80</td>
<td>m³</td>
<td>€ 3.92</td>
<td>€ 14.89</td>
</tr>
<tr>
<td>Clay supply and filling</td>
<td>6.80</td>
<td>m³</td>
<td>€ 19.31</td>
<td>€ 131.33</td>
</tr>
<tr>
<td>Finishing and seeding ground</td>
<td>25.00</td>
<td>m²</td>
<td>€ 2.00</td>
<td>€ 50.00</td>
</tr>
<tr>
<td>Safety and usage objects</td>
<td>1.00</td>
<td>m</td>
<td>€ 25.31</td>
<td>€ 25.31</td>
</tr>
<tr>
<td>Maintenance path</td>
<td>1.00</td>
<td>m</td>
<td>€ 21.84</td>
<td>€ 21.84</td>
</tr>
</tbody>
</table>

**Subtotal execution costs** | € 2,263.89 |
Further detailing | 10.00% | € 226.39 |
**Direct execution costs** | € 2,490.27 |
| Execution costs | 7.00% | € 174.32 |
| Profit and risks | 5.00% | € 124.51 |
| **Total** | € 2,789.11 |

### Service Costs (per year)

<table>
<thead>
<tr>
<th>Description</th>
<th>Amount</th>
<th>Unit</th>
<th>Price/Unit</th>
<th>Price per M1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monthly inspection</td>
<td>1.00</td>
<td>/m/J</td>
<td>€ 1.50</td>
<td>€ 1.50</td>
</tr>
<tr>
<td>Mowing grass</td>
<td>10.00</td>
<td>m²</td>
<td>€ 0.39</td>
<td>€ 3.90</td>
</tr>
<tr>
<td><strong>Total</strong></td>
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<td>€ 5.40</td>
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### Service Costs (per 10 years)

<table>
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<tr>
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<th>Amount</th>
<th>Unit</th>
<th>Price/Unit</th>
<th>Price per M1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seeding once every 10 year</td>
<td>2.50</td>
<td>m²</td>
<td>€ 0.45</td>
<td>€ 1.13</td>
</tr>
<tr>
<td><strong>Total</strong></td>
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<td></td>
<td></td>
<td>€ 1.13</td>
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</tbody>
</table>

### Reactive Maintenance (per year)

<table>
<thead>
<tr>
<th>Description</th>
<th>Amount</th>
<th>Unit</th>
<th>Price/Unit</th>
<th>Price per M1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Land site preparation</td>
<td>4.80</td>
<td>m²</td>
<td>€ 3.00</td>
<td>€ 14.40</td>
</tr>
<tr>
<td>Remove non water retaining objects</td>
<td>4.80</td>
<td>m²</td>
<td>€ 1.00</td>
<td>€ 4.80</td>
</tr>
<tr>
<td>Installation reinforced concrete capping beam</td>
<td>0.10</td>
<td>m³</td>
<td>€ 1,129.44</td>
<td>€ 112.94</td>
</tr>
<tr>
<td>Installation reinforced concrete wall</td>
<td>0.30</td>
<td>m³</td>
<td>€ 719.19</td>
<td>€ 215.76</td>
</tr>
<tr>
<td><strong>Subtotal execution costs</strong></td>
<td></td>
<td></td>
<td></td>
<td>€ 132.14</td>
</tr>
</tbody>
</table>
Further detailing | 10.00% | | € 13.21 |
| **Direct execution costs** | € 145.36 |
| Execution costs | 7.00% | | € 10.18 |
| Profit and risks | 5.00% | | € 7.27 |
| **Total** | € 162.80 |

### End-of-Life Costs

<table>
<thead>
<tr>
<th>Description</th>
<th>Amount</th>
<th>Unit</th>
<th>Price/Unit</th>
<th>Price per M1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Demolish concrete capping beam</td>
<td>2.00</td>
<td>m</td>
<td>€ 17.69</td>
<td>€ 35.39</td>
</tr>
<tr>
<td>Demolish anchors</td>
<td>1.00</td>
<td>m</td>
<td>€ 22.32</td>
<td>€ 22.32</td>
</tr>
<tr>
<td>Soil excavation</td>
<td>16.50</td>
<td>m³</td>
<td>€ 3.73</td>
<td>€ 61.60</td>
</tr>
<tr>
<td>Remove sheet piles + recycling</td>
<td>15.00</td>
<td>m²</td>
<td>€ 21.30</td>
<td>€ 319.51</td>
</tr>
<tr>
<td>Remove maintenance path</td>
<td>3.00</td>
<td>m²</td>
<td>€ 10.00</td>
<td>€ 30.00</td>
</tr>
<tr>
<td><strong>Total</strong></td>
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<td>€ 468.82</td>
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### Alternative K: Project costs

<table>
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<tr>
<th>Project</th>
<th>Amount</th>
<th>Unit</th>
<th>Price/Unit</th>
<th>Price per m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>Land site preparation</td>
<td>15,00</td>
<td>m²</td>
<td>€ 3,00</td>
<td>€ 45,00</td>
</tr>
<tr>
<td>Remove non water retaining objects</td>
<td>8,00</td>
<td>m²</td>
<td>€ 1,00</td>
<td>€ 8,00</td>
</tr>
<tr>
<td>Soil excavating for foundation</td>
<td>3,00</td>
<td>m³</td>
<td>€ 3,75</td>
<td>€ 11,26</td>
</tr>
<tr>
<td>Remove partly present levee structure (masonry + concrete?)</td>
<td>3,00</td>
<td>m²</td>
<td>€ 18,88</td>
<td>€ 56,64</td>
</tr>
<tr>
<td>Installation of sheet pile AZ-26</td>
<td>1,00</td>
<td>m</td>
<td>€ 128,68</td>
<td>€ 128,68</td>
</tr>
<tr>
<td>Installation reinforced concrete floor</td>
<td>1,25</td>
<td>m³</td>
<td>€ 371,25</td>
<td>€ 464,06</td>
</tr>
<tr>
<td>Installation reinforced concrete capping beam</td>
<td>0,40</td>
<td>m³</td>
<td>€ 1,096,25</td>
<td>€ 438,50</td>
</tr>
<tr>
<td>Installation reinforced concrete wall</td>
<td>0,24</td>
<td>m³</td>
<td>€ 719,19</td>
<td>€ 172,61</td>
</tr>
<tr>
<td>Installation anchors</td>
<td>1,00</td>
<td>m</td>
<td>€ 271,64</td>
<td>€ 271,64</td>
</tr>
<tr>
<td>Installation of expansion joints</td>
<td>1,00</td>
<td>m²</td>
<td>€ 6,88</td>
<td>€ 6,88</td>
</tr>
<tr>
<td>Installation of glass wall</td>
<td>1,00</td>
<td>m²</td>
<td>€ 402,00</td>
<td>€ 402,00</td>
</tr>
<tr>
<td>Installation of glass wall</td>
<td>0,90</td>
<td>m²</td>
<td>€ 402,00</td>
<td>€ 361,80</td>
</tr>
<tr>
<td>Installation of glass wall</td>
<td>1,00</td>
<td>m²</td>
<td>€ 223,13</td>
<td>€ 223,13</td>
</tr>
<tr>
<td>Install anchors</td>
<td>1,00</td>
<td>m²</td>
<td>€ 21,64</td>
<td>€ 21,64</td>
</tr>
<tr>
<td>Install anchors</td>
<td>1,00</td>
<td>m²</td>
<td>€ 1,50</td>
<td>€ 1,50</td>
</tr>
<tr>
<td>Install anchors</td>
<td>1,00</td>
<td>m²</td>
<td>€ 1,87</td>
<td>€ 1,87</td>
</tr>
<tr>
<td>Install anchors</td>
<td>1,00</td>
<td>m²</td>
<td>€ 3,60</td>
<td>€ 3,60</td>
</tr>
<tr>
<td>Install anchors</td>
<td>1,00</td>
<td>m²</td>
<td>€ 223,13</td>
<td>€ 223,13</td>
</tr>
<tr>
<td>Install anchors</td>
<td>1,00</td>
<td>m²</td>
<td>€ 223,13</td>
<td>€ 223,13</td>
</tr>
<tr>
<td>Install anchors</td>
<td>1,00</td>
<td>m²</td>
<td>€ 223,13</td>
<td>€ 223,13</td>
</tr>
</tbody>
</table>

### Subtotal execution costs

- **Further detailing**: 10,00% | € 432,30
- **Direct execution costs**: 10,00% | € 4,755,33
- **Execution costs**: 7,00% | € 332,87
- **Profit and risks**: 5,00% | € 237,77
- **Total**: € 5,325,97

### Service costs (per year)

- **Monthly inspection**: 1,00 /m/J | € 1,50
- **Mowing grass**: 4,80 m² | € 0,39
- **Glass cleaning at both sides per year**: 1,00 m² | € 3,60
- **Total**: € 6,97

### Service costs (per 10 years)

- **Seeding once every 10 year**: 4,80 m² | € 0,45
- **Reinstallation of rubbers and kit**: 1,00 m | € 15,33
- **Total**: € 17,49

### Reactive maintenance (per year)

- **Broken glass (1m² every 200 m³ per year)**: 0,01 m² | € 625,13
- **Total**: € 3,13

### Major Maintenance (per 50 years)

- **Remove existing glass wall**: 1,00 m | € 13,93
- **Installation of glass wall**: 1,00 m² | € 402,00
- **Total**: € 237,06

### Subtotal execution costs

- **Further detailing**: 10,00% | € 23,71
- **Direct execution costs**: 10,00% | € 260,76
- **Execution costs**: 7,00% | € 18,25
- **Profit and risks**: 5,00% | € 13,04
- **Total**: € 292,06

### End-of-Life Costs

- **Remove existing glass wall**: 1,00 m | € 13,93
- **Demolish concrete capping beam**: 1,00 m | € 28,13
- **Demolish concrete floor**: 1,25 m³ | € 68,40
- **soil excavation**: 4,00 m³ | € 7,37
- **Demolish anchors**: 1,00 m | € 53,18
- **Remove sheet piles + recycling**: 9,00 m² | € 21,50
- **Remove maintenance path**: 3,00 m² | € 10,00
- **Total**: € 419,17
### DETERMINISTIC WITHOUT OPTIMIZATION

<table>
<thead>
<tr>
<th>Alternative</th>
<th>I_tot (€)</th>
<th>B (€)</th>
<th>NPV (€)</th>
<th>Sequence</th>
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<tbody>
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(with no extra robustness)

### DETERMINISTIC WITH 0.60 METER ROBUSTNESS

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### PROBABILISTIC WITH OPTIMIZATION

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## SECTION 3:
### Alternative B:

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<td>1,00</td>
<td>m²</td>
<td>€ 105,82</td>
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<tr>
<td>Reinforcing concrete capping beam</td>
<td>0,15</td>
<td>m³</td>
<td>€ 1,129,44</td>
<td>€ 169,42 H1</td>
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<td><strong>Subtotal execution costs</strong></td>
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<td>Further detailing</td>
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<td>Execution costs</td>
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<tr>
<td>Profit and risks</td>
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</table>

### Service costs (per year)

| Service costs (per year) | | | | | |
|--------------------------|--------|------|------------|--------------|
| Monthly inspection | 1,00 | /m/J | € 1,50 | € 1,50 C |
| Use of demountable system | 1,00 | m | € 2,53 | € 25,42 C |
| **Total** | | | **€ 26,92** |

### Reactive maintenance (t=25)

| Reactive maintenance (t=25) | | | | | |
|-----------------------------|--------|------|------------|--------------|
| Installation due to high water | 1,00 | m² | € 18,75 | € 18,75 |
| **Total** | | | **€ 18,75** |

### Major Maintenance (per 50 years)

| Major Maintenance (per 50 years) | | | | | |
|----------------------------------|--------|------|------------|--------------|
| Supplying additional | 1,00 | m | € 275,00 | € 275,00 H2 |
| Increasing stock ability (1/3 of present stock costs) | 1,00 | m² | € 105,82 | € 105,82 H2 |
| Reinforcing concrete capping beam | 0,15 | m³ | € 1,129,44 | € 169,42 C |
| **Subtotal execution costs** | | | **€ 380,82** |
| Further detailing | | | 10,00% | € 38,08 |
| **Direct execution costs** | | | **€ 418,90** |
| Execution costs | | | 7,00% | € 29,32 |
| Profit and risks | | | 5,00% | € 20,95 |
| **Total** | | | **€ 469,17** |

### End-of-Life Costs

| End-of-Life Costs | | | | | |
|-------------------|--------|------|------------|--------------|
| Demolish concrete capping beam | 1,00 | m | € 28,13 | € 28,13 C |
| Remove demountable system | 50,00 | kg/m | -€ 0,80 | -€ 40,00 C |
| **Total** | | | **-€ 11,88** |
### Project costs

<table>
<thead>
<tr>
<th>Description</th>
<th>Amount</th>
<th>Unit</th>
<th>Price/Unit</th>
<th>Price per m²</th>
<th>V/C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Land site preparation</td>
<td>7,00</td>
<td>m²</td>
<td>€ 3,00</td>
<td>€ 21,00</td>
<td>C</td>
</tr>
<tr>
<td>Demolisch concrete</td>
<td>0,50</td>
<td>m²</td>
<td>17,69</td>
<td>€ 8,85</td>
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</tr>
<tr>
<td>Installation reinforced concrete wall</td>
<td>0,20</td>
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<td>€ 719,19</td>
<td>€ 143,84</td>
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<tr>
<td>Soil excavating</td>
<td>1,00</td>
<td>m³</td>
<td>€ 3,75</td>
<td>€ 3,75</td>
<td>C</td>
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<td>Restore soil</td>
<td>1,00</td>
<td>m³</td>
<td>€ 3,92</td>
<td>€ 3,92</td>
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</tr>
<tr>
<td>Installation of glass wall</td>
<td>1,00</td>
<td>m²</td>
<td>€ 402,00</td>
<td>€ 402,00</td>
<td>H1</td>
</tr>
<tr>
<td>Installation of glass wall</td>
<td>0,90</td>
<td>m²</td>
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<td>/m</td>
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<tr>
<td>Maintenance path</td>
<td>1,00</td>
<td>m</td>
<td>€ 21,84</td>
<td>€ 21,84</td>
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</tbody>
</table>

Subtotal execution costs € 788,12

Further detailing 10,00% € 78,81

Direct execution costs € 866,93

Execution costs 7,00% € 60,69

Profit and risks 5,00% € 43,35

Total € 970,96

### Service costs (per year)

<table>
<thead>
<tr>
<th>Description</th>
<th>Amount</th>
<th>Unit</th>
<th>Price/Unit</th>
<th>Price per m²</th>
<th>V/C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monthly inspection</td>
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<td>€ 3,60</td>
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### Service costs (per 10 years)

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<th>V/C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinstallation of rubbers and kit</td>
<td>1,00</td>
<td>m</td>
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<td>€ 15,33</td>
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</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td></td>
<td>€ 15,33</td>
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</table>

### Reactive maintenance (per year)

<table>
<thead>
<tr>
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<th>Amount</th>
<th>Unit</th>
<th>Price/Unit</th>
<th>Price per m²</th>
<th>V/C</th>
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</thead>
<tbody>
<tr>
<td>Broken glass (1m² every 200 m³ per year)</td>
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<td>m²</td>
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### Major Maintenance (per 50 years)

<table>
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<tr>
<th>Description</th>
<th>Amount</th>
<th>Unit</th>
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<th>Price per m²</th>
<th>V/C</th>
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</thead>
<tbody>
<tr>
<td>Remove existing glass wall</td>
<td>1,00</td>
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<tr>
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<td>m²</td>
<td>€ 402,00</td>
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<tr>
<td>Installation of glass wall</td>
<td>1,00</td>
<td>m²</td>
<td>€ 223,13</td>
<td>€ 223,13</td>
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</table>

Subtotal execution costs € 237,06

Further detailing 10,00% € 23,71

Direct execution costs € 260,76

Execution costs 7,00% € 18,25

Profit and risks 5,00% € 13,04

Total € 292,06

### End-of-Life Costs

<table>
<thead>
<tr>
<th>Description</th>
<th>Amount</th>
<th>Unit</th>
<th>Price/Unit</th>
<th>Price per m²</th>
<th>V/C</th>
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<tbody>
<tr>
<td>Remove existing glass wall</td>
<td>1,00</td>
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### Alternative F:

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<td>Installation reinforced concrete wall</td>
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<tr>
<td>Installation due to high water</td>
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<td></td>
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<td><strong>Major Maintenance (per 50 years)</strong></td>
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<tr>
<td>Installation reinforced concrete wall</td>
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<td>m³</td>
<td>€ 719,19</td>
<td>€ 431,51</td>
<td>C</td>
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## Alternative L:

### Project costs

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<tr>
<td>Installation reinforced concrete wall</td>
<td>0,48 m³</td>
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<td>€ 342.67</td>
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<td>Installation of expansion joints</td>
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<td>€ 21.84</td>
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### Subtotal execution costs

- **€ 1,699.44**
- **Further detailing 10,00% € 169.94**
- **Direct execution costs € 1,869.38**
- **Execution costs 7.00% € 130.86**
- **Profit and risks 5.00% € 93.47**
- **Total € 2,093.70**

### Service costs (per year)

- **Monthly inspection 1.00 /m/J € 1.50 € 1.50 C**
- **Total € 1.50**

### Major Maintenance (per 50 years)

<table>
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<tr>
<th>Description</th>
<th>Amount</th>
<th>Unit</th>
<th>Price/Unit</th>
<th>Price per m³</th>
<th>V/C</th>
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<tr>
<td>Land site preparation</td>
<td>10,00 m²</td>
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<td>Demolish road</td>
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<td>€ 79.41</td>
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<td>Soil excavating</td>
<td>4,76 m³</td>
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### Subtotal execution costs

- **€ 2,522.00**
- **Further detailing 10.00% € 252.20**
- **Direct execution costs € 2,774.20**
- **Execution costs 7.00% € 194.19**
- **Profit and risks 5.00% € 138.71**
- **Total € 3,107.11**

### End-of-Life Costs

<table>
<thead>
<tr>
<th>Description</th>
<th>Amount</th>
<th>Unit</th>
<th>Price/Unit</th>
<th>Price per m³</th>
<th>V/C</th>
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### Total

- **€ 338.92**
## Alternative B:

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### Subtotal execution costs

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### Service costs (per year)

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### Reactive maintenance (t=25)

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### Major Maintenance (per 50 years)

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<tr>
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<td>m³</td>
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### End-of-Life Costs

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<td>1,00</td>
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<td>€ 21,84</td>
<td>€ 21,84</td>
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</tbody>
</table>

**Subtotal execution costs** \( \text{€ 989,12} \)

**Direct execution costs** \( \text{€ 1.088,03} \)

**Execution costs** 7,00% \( \text{€ 76,16} \)

**Profit and risks** 5,00% \( \text{€ 54,40} \)

**Total** \( \text{€ 1.218,60} \)

**Service costs (per year)**

| Monthly inspection               | 1,00 | /m/1 | € 1,50       | € 1,50       | C   |
| Glass cleaning at both sides per year | 1,00 | m2   | € 3,60       | € 3,60       | H1, H2 |
| **Total**                         |      |      | € 5,10       |              |     |

**Service costs (per 10 years)**

| Reinstallation of rubbers and kit | 1,00 | m   | € 15,33       | € 15,33       | C   |
| **Total**                         |      |    | € 15,33       |              |     |

**Reactive maintenance (per year)**

| Broken glass (1m2 every 200 m1 per year) | 0,01 | m2   | € 625,13       | € 3,13       | H1, H2 |
| **Total**                               |      |       |                | € 3,13       |

**Major Maintenance (per 50 years)**

| Remove existing glass wall | 1,00 | m   | € 13,93       | € 13,93       | C   |
| Installation of glass wall | 1,00 | m2  | € 402,00      | € 402,00      | H2  |
| Installation of glass wall | 1,00 | m2  | € 223,13      | € 223,13      | C   |
| **Subtotal execution costs**          |      |     | € 237,06      |              |     |

**Direct execution costs** \( \text{€ 260,76} \)

**Execution costs** 7,00% \( \text{€ 18,25} \)

**Profit and risks** 5,00% \( \text{€ 13,04} \)

**Total** \( \text{€ 292,06} \)

**End-of-Life Costs**

| Remove existing glass wall | 1,00 | m   | € 13,93       | € 13,93       | C   |
| Demolisch concrete capping beam | 1,00 | m   | € 28,13       | € 28,13       | C   |
| **Total**                  |      |     | € 42,06       |              |     |
### Alternative D: Project costs

<table>
<thead>
<tr>
<th>Project costs</th>
<th>amount</th>
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<th>V/C</th>
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</thead>
<tbody>
<tr>
<td>Land site preparation</td>
<td>7,00</td>
<td>m²</td>
<td>€ 3,00</td>
<td>€ 21,00</td>
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<tr>
<td>Replace trees</td>
<td>0,25</td>
<td>m³</td>
<td>€ 252,70</td>
<td>€ 63,18</td>
<td>C</td>
</tr>
<tr>
<td>Demolish concrete</td>
<td>0,50</td>
<td>m³</td>
<td>€ 17,69</td>
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<tr>
<td>Installation reinforced concrete wall</td>
<td>0,20</td>
<td>m³</td>
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<td>C</td>
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<tr>
<td>Soil excavating</td>
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<tr>
<td>Installation of expansion joints</td>
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<td>C</td>
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<td>Finishing and seeding ground</td>
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<tr>
<td>Maintenance path</td>
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### Service costs (per year)

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<tr>
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<tr>
<td>Mowing grass</td>
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<td><strong>Total</strong></td>
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### Service costs (per 10 years)

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<th>Service costs (per 10 years)</th>
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<tr>
<td>Seeding once every 10 year</td>
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### Reactive maintenance (per year)

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<th>amount</th>
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<tbody>
<tr>
<td><strong>Total</strong></td>
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<td></td>
<td></td>
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### End-of-Life Costs

<table>
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<tr>
<th>End-of-Life Costs</th>
<th>amount</th>
<th>unit</th>
<th>price/unit</th>
<th>price per m³</th>
<th>V/C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Remove soil</td>
<td>12,00</td>
<td>m³</td>
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<td>€ 45,02</td>
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<tr>
<td>Remove concrete wall</td>
<td>1,00</td>
<td>m</td>
<td>€ 17,69</td>
<td>€ 17,69</td>
<td>C</td>
</tr>
<tr>
<td>Remove maintenance path</td>
<td>3,00</td>
<td>m²</td>
<td>€ 10,00</td>
<td>€ 30,00</td>
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<tr>
<td><strong>Total</strong></td>
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### Alternative F:

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<th>V/C</th>
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<tr>
<td>Land site preparation</td>
<td>7.00</td>
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<tr>
<td>Demolish concrete</td>
<td>1.00</td>
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<td>17,69</td>
<td>C</td>
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<tr>
<td>Replace trees</td>
<td>0,25</td>
<td>m1</td>
<td>€ 252,70</td>
<td>€ 63,18</td>
<td>C</td>
</tr>
<tr>
<td>Installation reinforced concrete wall</td>
<td>0,60</td>
<td>m3</td>
<td>€ 719,19</td>
<td>€ 431,51</td>
<td>C</td>
</tr>
<tr>
<td>Installation reinforced concrete wall</td>
<td>0,60</td>
<td>m3</td>
<td>€ 719,19</td>
<td>€ 431,51</td>
<td>H1</td>
</tr>
<tr>
<td>Installation of expansion joints</td>
<td>1,00</td>
<td>m</td>
<td>€ 6,88</td>
<td>€ 6,88</td>
<td>C</td>
</tr>
<tr>
<td>Soil excavating</td>
<td>1,00</td>
<td>m3</td>
<td>€ 3,75</td>
<td>€ 3,75</td>
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<tr>
<td>Installation reinforced concrete floor</td>
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<td>m3</td>
<td>€ 336,34</td>
<td>€ 269,07</td>
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</tr>
<tr>
<td>Installion reinforced concrete floor</td>
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<td>m3</td>
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<td>Restore soil</td>
<td>1,00</td>
<td>m3</td>
<td>€ 3,92</td>
<td>€ 3,92</td>
<td>C</td>
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<tr>
<td>Apply soil + supply</td>
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<td>€ 21,84</td>
<td>C</td>
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<tr>
<td>Demountable system</td>
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<td>m</td>
<td>€ 1.400,29</td>
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<tr>
<td>Demountable system</td>
<td>1,00</td>
<td>m</td>
<td>€ 1.400,29</td>
<td>€ 1.400,29</td>
<td>H1</td>
</tr>
</tbody>
</table>

**Subtotal execution costs**

- **Further detailing**
  - 10,00%
  - € 239,22

**Direct execution costs**

- **Execution costs**
  - 7,00%
  - € 184,20

- **Profit and risks**
  - 5,00%
  - € 131,57

**Total**

- € 2,947,23

### Service costs (per year)

- **Monthly inspection**
  - 1,00 /m/J
  - € 1,50

- **Use of dynamic system (testing)**
  - 1,00 m
  - € 2,19

**Total**

- € 3,69

### Service costs (per 10 years)

- **Reinstalation of rubbers and kit**
  - 1,00 m
  - € 15,33

- **Painting**
  - 1,00 m
  - € 50,00

**Total**

- € 65,33

### Reactive maintenance (t=25)

- **Installation due to high water**
  - 1,00 m2
  - € 4,04

**Total**

- € 4,04

### Major Maintenance (per 50 years)

- **Land site preparation**
  - 4,80 m2
  - € 3,00

- **Demolish concrete**
  - 0,30 m3
  - € 719,19

- **Installation reinforced concrete wall**
  - 0,60 m3
  - € 719,19

- **Installation reinforced concrete wall**
  - 0,60 m3
  - € 719,19

- **Installation reinforced concrete floor**
  - 0,80 m3
  - € 336,34

- **Installation reinforced concrete floor**
  - 0,40 m3
  - € 125,00

- **Demountable system**
  - 1,00 m
  - € 1.400,29

- **Demountable system**
  - 1,00 m
  - € 1.400,29

- **Installation of expansion joints**
  - 1,00 m
  - € 6,88

- **Take away and store maintenance path**
  - 3,00 m2
  - € 0,45

- **Soil excavating + supply**
  - 3,00 m3
  - € 10,82

- **Installation of maintenance path**
  - 1,00 m2
  - € 21,84

**Subtotal execution costs**

- **Further detailing**
  - 10,00%
  - € 219,84

**Direct execution costs**

- **Execution costs**
  - 7,00%
  - € 169,28

- **Profit and risks**
  - 5,00%
  - € 120,91

**Total**

- € 2,708,45

### End-of-Life Costs

- **Demolish concrete capping beam**
  - 3,00 m
  - € 17,69

- **Remove maintenance path**
  - 3,00 m2
  - € 10,00

- **Remove dynamic system**
  - 1,00 m
  - € 9,81

**Total**

- € 92,89
### Project costs

<table>
<thead>
<tr>
<th>Description</th>
<th>Amount</th>
<th>Unit</th>
<th>Price/Unit</th>
<th>Price per m²</th>
<th>V/C</th>
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</thead>
<tbody>
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<td>0.25</td>
<td>m</td>
<td>€ 252.70</td>
<td>€ 63.18</td>
<td>C</td>
</tr>
<tr>
<td>Demolish concrete</td>
<td>1.00</td>
<td>m²</td>
<td>€ 17.69</td>
<td>€ 17.69</td>
<td>C</td>
</tr>
<tr>
<td>Soil excavating</td>
<td>5.00</td>
<td>m³</td>
<td>€ 3.75</td>
<td>€ 18.76</td>
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<td>€ 167.02</td>
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<tr>
<td>Installation reinforced concrete wall</td>
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<td>m³</td>
<td>€ 719.19</td>
<td>€ 143.84</td>
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<tr>
<td>Restore soil</td>
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<td>Clay supply and filling</td>
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<td>m³</td>
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<tr>
<td>Installation reinforced concrete wall</td>
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<td>m³</td>
<td>€ 719.19</td>
<td>€ 215.76</td>
<td>H1</td>
</tr>
<tr>
<td>Installation of expansion joints</td>
<td>1.00</td>
<td>m</td>
<td>€ 6.88</td>
<td>€ 6.88</td>
<td>C</td>
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<tr>
<td>Finishing and seeding ground</td>
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<td>m²</td>
<td>€ 2.00</td>
<td>€ 20.00</td>
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<td>Maintenance path</td>
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<td>€ 21.84</td>
<td>€ 21.84</td>
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<td>Reinstallation of cables and pipes</td>
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</table>

**Subtotal execution costs**: € 3,130.35

*Further detailing*: 10.00%  
*Direct execution costs*: € 3,443.39

### Service costs (per year)

<table>
<thead>
<tr>
<th>Description</th>
<th>Amount</th>
<th>Unit</th>
<th>Price/Unit</th>
<th>Price per m²</th>
<th>V/C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monthly inspection</td>
<td>1.00</td>
<td>/m/J</td>
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<td>C</td>
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**Total**: € 5.40

### Service costs (per 10 years)

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<th>Price per m²</th>
<th>V/C</th>
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</thead>
<tbody>
<tr>
<td>Seeding once every 10 year</td>
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**Total**: € 4.50

### Reactive maintenance (per year)

<table>
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<th>Description</th>
<th>Amount</th>
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<th>Price/Unit</th>
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<th>V/C</th>
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</thead>
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<td>remove non water retaining objects</td>
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<td>m²</td>
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<td>€ 4.80</td>
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<td>0.10</td>
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**Subtotal execution costs**: € 154.65

*Further detailing*: 10.00%  
*Direct execution costs*: € 170.12

### End-of-Life Costs

<table>
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<th>Description</th>
<th>Amount</th>
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<th>Price per m²</th>
<th>V/C</th>
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</thead>
<tbody>
<tr>
<td>Remove soil</td>
<td>12.00</td>
<td>m³</td>
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<td>€ 45.02</td>
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<tr>
<td>Remove concrete wall</td>
<td>1.00</td>
<td>m</td>
<td>€ 17.69</td>
<td>€ 17.69</td>
<td>C</td>
</tr>
<tr>
<td>Remove sheet piles + recycling</td>
<td>15.00</td>
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<tr>
<td>Remove maintenance path</td>
<td>3.00</td>
<td>m²</td>
<td>€ 10.00</td>
<td>€ 30.00</td>
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</tr>
</tbody>
</table>

**Total**: € 412.22

### Additional notes:

- **Profit and risks**: 5.00%  
- **Total**: € 3,856.60

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*Note: All costs are in euros.*
### Deterministic Without Optimization

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<th>NPV</th>
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<td>1</td>
<td>Alternative B</td>
<td>€ 21,343.176</td>
<td>€ 19,751.088</td>
<td>-€ 1,592.088</td>
</tr>
<tr>
<td>2</td>
<td>Alternative E</td>
<td>€ 21,384.494</td>
<td>€ 19,751.088</td>
<td>-€ 1,633.406</td>
</tr>
<tr>
<td>3</td>
<td>Alternative F</td>
<td>€ 21,945.777</td>
<td>€ 19,751.088</td>
<td>-€ 2,194.689</td>
</tr>
<tr>
<td>4</td>
<td>Alternative L</td>
<td>€ 21,673.497</td>
<td>€ 18,520.590</td>
<td>-€ 3,152.907</td>
</tr>
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</table>

### Deterministic with 0.80 Meter Robustness

<table>
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<tr>
<th>Sequence</th>
<th>Alternative</th>
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<th>B</th>
<th>NPV</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>Alternative B</td>
<td>€ 24,681.790</td>
<td>€ 55,682.062</td>
<td>€ 31,000.272</td>
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<tr>
<td>2</td>
<td>Alternative E</td>
<td>€ 24,707.057</td>
<td>€ 55,682.062</td>
<td>€ 30,975.005</td>
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<td>3</td>
<td>Alternative F</td>
<td>€ 25,416.822</td>
<td>€ 55,682.062</td>
<td>€ 30,265.240</td>
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<tr>
<td>4</td>
<td>Alternative L</td>
<td>€ 25,234.820</td>
<td>€ 55,471.995</td>
<td>€ 30,237.175</td>
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</table>

### Probabilistic With Optimization

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<tbody>
<tr>
<td>1</td>
<td>Alternative B</td>
<td>€ 24,326.084</td>
<td>€ 55,682.062</td>
<td>€ 31,355.978</td>
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<td>2</td>
<td>Alternative E</td>
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<td>3</td>
<td>Alternative F</td>
<td>€ 25,048.646</td>
<td>€ 55,682.062</td>
<td>€ 30,633.416</td>
</tr>
<tr>
<td>4</td>
<td>Alternative L</td>
<td>€ 24,875.161</td>
<td>€ 55,471.995</td>
<td>€ 30,596.835</td>
</tr>
</tbody>
</table>

### List of Alternatives

- **3 Alternative B**: € 21,343.176, € 19,751.088, -€ 1,592.088
- **3 Alternative E**: € 21,384.494, € 19,751.088, -€ 1,633.406
- **3 Alternative F**: € 21,945.777, € 19,751.088, -€ 2,194.689
- **3 Alternative L**: € 21,673.497, € 18,520.590, -€ 3,152.907
- **4 Alternative B**: € 21,458.091, € 19,751.088, -€ 1,707.003
- **4 Alternative C**: € 21,521.878, € 19,751.088, -€ 1,770.791
- **4 Alternative D**: € 21,457.136, € 19,751.088, -€ 1,706.048
- **4 Alternative F**: € 21,954.169, € 19,751.088, -€ 2,203.082
- **4 Alternative I**: € 21,845.092, € 19,751.088, -€ 2,094.005
- **3 Alternative B**: € 24,681.790, € 55,682.062, € 31,000.272
- **3 Alternative E**: € 24,707.057, € 55,682.062, € 30,975.005
- **3 Alternative F**: € 25,416.822, € 55,682.062, € 30,265.240
- **3 Alternative L**: € 25,234.820, € 55,471.995, € 30,237.175
- **4 Alternative B**: € 24,799.643, € 55,682.062, € 30,882.419
- **4 Alternative C**: € 24,869.378, € 55,682.062, € 30,812.684
- **4 Alternative D**: € 24,784.823, € 55,682.062, € 30,897.239
- **4 Alternative F**: € 25,396.993, € 55,682.062, € 30,285.069
- **4 Alternative I**: € 25,165.015, € 55,682.062, € 30,517.047
- **3 Alternative B**: € 24,326.084, € 55,682.062, € 31,355.978
- **3 Alternative E**: € 24,348.520, € 55,682.062, € 31,333.542
- **3 Alternative F**: € 25,048.646, € 55,682.062, € 30,633.416
- **3 Alternative L**: € 24,875.161, € 55,471.995, € 30,596.835
- **4 Alternative B**: € 24,447.442, € 55,682.062, € 31,234.620
- **4 Alternative C**: € 24,518.320, € 55,682.062, € 31,163.742
- **4 Alternative D**: € 24,434.536, € 55,682.062, € 31,247.526
- **4 Alternative F**: € 25,024.560, € 55,682.062, € 30,657.502
- **4 Alternative I**: € 24,812.419, € 55,682.062, € 30,869.643