Room for the Riverward Dike

Assessing the hydraulic impact, feasibility and cost-effectiveness of outward dike reinforcement under relaxed regulations with a case study of the Waal

Master Thesis

Koen van Eeuwijk

TU Delft

Haskoning
Enhancing Society Together

Room for the Riverward Dike

Assessing the hydraulic impact, feasibility and cost-effectiveness of outward dike reinforcement under relaxed regulations with a case study of the Waal

by

Koen van Eeuwijk

To obtain the degree of Master of Science
at the Delft University of Technology,
to be defended publicly on Monday November 3, 2025 at 16:00.

Student number:	4865480	
Thesis committee:	Dr. Ir. C. Mai Van,	TU Delft, supervisor
	Dr. Ir. E. Mosselman,	TU Delft, supervisor
	Ir. M.A. Schoemaker,	TU Delft & Haskoning, company supervisor

This Master's thesis is in collaboration with Haskoning

Cover image: Jan Bouwhuis, via De Tielenaar, August 2023

Preface

This master's thesis is the result of research carried out in collaboration with Haskoning and Delft University of Technology. With it, I complete my Master's degree in Hydraulic Engineering and bring to a close my seven-year journey at TU Delft.

I arrived in Delft keen to explore as much as possible, with little idea of where I wanted my studies to lead or what I wanted to become. The combination of design and technical analysis had always fascinated me, and Civil Engineering soon proved to be the perfect match. Choosing between soil and water was perhaps one of the hardest decisions of my studies. I'm very glad I entered the field of Hydraulic Engineering, while still being able to draw on the soil mechanics I had come to know.

This thesis reflects the interplay of my main interests: water and soil mechanics, landscape dynamics, and societal impact. It explores outward dike expansion that absorbs its own hydraulic effects as an additional design alternative for flood defences. In doing so, dike reinforcement projects may become less complex, less costly, and less disruptive for local communities.

I would like to sincerely thank my supervisors, Cong Mai Van, Erik Mosselman, and Maarten Schoemaker, for their invaluable guidance and constructive, ongoing feedback throughout this project. In particular, I am grateful to Cong for insisting on monthly presentations and feedback sessions, which allowed me to refine the project scope in a timely manner. To Erik, for his reassurances and advice that this thesis need not aim for the most high-tech innovations and for always taking the time to comment on my progress and laments at the end of each Week. Guidance that helped me greatly in the early stages. And to Maarten, for introducing me to Haskoning, equipping me with practical tools from the dike reinforcement field, investing time to help me improve the structure and clarity of my thesis, and for our nice talks during our walks.

I also wish to extend a special thanks to all the experts at Haskoning. They not only made time for interviews and questions, but consistently showed great enthusiasm for my topic and a willingness to support me. Even more than I would have dared to ask. Without their input, this thesis could never have reached its current level of practical application. From the very start, I felt welcome at Haskoning, and I am grateful to have met and interacted with so many colleagues, both within and beyond my team.

Finally, I would like to thank my parents, my brothers, the girlfriend, close friends, study friends, old friends and all the other amazing people I have encountered during my time in Delft, who have each contributed in their own way to shaping the person I am glad to have become. These people made my time as a student in Delft an absolute blast.

Koen van Eeuwijk
Rotterdam, September 2025

Abstract

Under the Dutch Flood Protection Programme (HWBP), extensive reinforcement of primary flood defences is planned by 2050. However, progress is slow and costs are exceeding expectations. Current reinforcement strategies face significant challenges: inward reinforcement is constrained by spatial limitations, while construction-based solutions are costly and lack flexibility in light of climate policy uncertainty. Outward dike reinforcement (ODR) offers an alternative, but its application is currently restricted by regulations under the Rivierkundig Beoordelingskader (RBK). In particular, the '1-mm rule' requires that any hydraulic impact be mitigated to within 1 mm, often necessitating substantial mitigation works that render ODR disproportionately complex and expensive, and thus rarely applied despite its promise.
This thesis investigates whether ODR can be considered a technically feasible and cost-effective alternative under a relaxed RBK framework, in which its hydraulic impact is absorbed by the flood defence system, eliminating the need for mitigation measures. In doing so, it aims to support engineering practice through preliminary design insights, while also identifying regulatory elements that may warrant reconsideration to facilitate broader applicability of ODR.

Four self-designed conceptual dike variants are created to assess the hydraulic impact, technical feasibility, and cost-effectiveness of ODR under varying conditions. The designs are developed at a preliminary level and represent inward, construction-based, and two outward variants with 5-metre (Tuimeldijk) and 20-metre expansions, applied to a hypothetical reinforcement of the southern Waal banks. The outward variants define a bandwidth for evaluating the effects of expansion, providing a framework for generalising ODR performance. The hydraulic impact is analysed using a simplified 1D compound channel model to identify influential parameters and general trends, complemented by detailed D-Hydro Suite simulations for site-specific effects. These results inform the feasibility assessment, which uses exceedance-based limit-state formulations of the main failure mechanisms to evaluate dike performance and potential functional lifetime reduction (FLR). Cost-effectiveness is assessed through short-term design comparison and long-term net present value (NPV), using a life cycle cost (LCC) analysis that incorporates adaptability and feasibility outcomes.

The expected water level difference (WLD) due to ODR in the Waal River ranges from 0 to 4 cm, depending on the application and floodplain characteristics. Two design graphs are presented, offering first-order estimates of WLD for 5-metre and 20-metre expansions. The relationship between expansion and WLD is observed to be approximately linear, enabling interpolation across the design graphs. Bottlenecks in the river system show the highest WLD sensitivity, as narrow and smooth floodplains amplify hydraulic impact. Continuous reinforcement results in higher WLD than discrete interventions, although short interventions are penalised by steep local water level gradients and cause larger WLD per unit of intervention length. Adaptation lengths required to absorb WLD effects span several tens of kilometres, but depend strongly on endpoint definition.

ODR is technically feasible considering the performance of affected dikes. Of the main failure mechanisms, only overflow and piping are influenced by the hydraulic impact of ODR, resulting in an estimated FLR of approximately two years if applied in the Waal. Dike stability is never compromised while WLD remains below 10 centimetres, and effects on overtopping can be neglected. FLR is primarily governed by the annual climate-change-induced increase in hydraulic load, the magnitude of WLD, dike subsidence (for overflow), and the additional robustness provided by the blanket layer or sheet pile (for piping). If these structural elements include a buffer equal to or half the WLD, respectively, FLR due to piping is unlikely. Two strategies are considered to address FLR: adding an asphalt layer to all affected dikes or accepting the reduced lifetime.

The cost-effectiveness of an optimised ODR design is comparable to, and under certain scenarios, proves more cost-effective than construction-based reinforcement over a 100-year horizon, while offering greater adaptability where inward reinforcement is either unfeasible or prohibitively expensive. This is particularly true when recycled soil is used and in light of uncertainties regarding spatial constraints on the inner slope, sheet pile prices, and functional lifetime. The optimised design should minimise outward extent and material use, in line with minimal designs such as the Tuimeldijk variant, while ensuring sufficient resistance against piping without requiring sheet piles. It should be applied over stretches exceeding 10 kilometres, as cost-effectiveness increases with length. Choosing FLR proves to be the more cost-effective strategy of the two, as uncertainties associated with FLR have limited impact on overall cost-effectiveness. In short-term cost comparisons, ODR is only cost-effective if the design closely resembles the Tuimeldijk variant or incorporates recycled soil. Otherwise, the excessive soil volumes make it less competitive than non-outward alternatives.

These findings demonstrate that ODR, where the flood defence system absorbs the hydraulic impact, can be a technically feasible and cost-effective alternative to conventional reinforcement strategies, provided that RBK regulations are relaxed. It is therefore recommended to integrate this variant of ODR as the final step in the current line of reasoning for riverward reinforcement upheld by the HWBP. To enable practical application, it is proposed to relax the hydraulic limit in the RBK to 2 centimetres. This adjustment would allow a broader range of ODR configurations to be assessed using the presented design graphs, while maintaining cost-effectiveness and feasibility. Additionally, the 1-mm rule should serve as a threshold for defining adaptation length, provided a clear backwater formulation is integrated into the RBK. Further research is required to confirm that other regulatory conditions within the RBK are upheld under this revised hydraulic limit. This would allow ODR to serve as an additional alternative in complex situations, helping to reduce project complexity today while preserving flexibility for future challenges.

Contents

List of Figures

List of Tables

List of Abbreviations

BGR	Beleidslijn Grote Rivieren (Policy guideline for the major rivers)
DWL	Design water level
FLR	Functional lifetime reduction
HBN	Hydraulisch belasting niveau (Hydraulic load level)
HWBP	Hoogwaterbeschermingsprogramma (Flood Protection Programme)
KS ZSS	Kennisprogramma Zeespiegelstijging (Sea-level rise knowledge program)
LCC	Life cycle cost
NAP	Normaal Amsterdams Peil
NPV	Net present value
ODR	Outward dike reinforcement
RBK	Rivierkundig Beoordelingskader versie 6 (Assessment framework for the major rivers)
ULS	Ultimate limit state
WLD	Water level difference

1
Introduction

1.1. Background: shifting away from outward dike reinforcement

The Netherlands is a country shaped and governed by water. Water serves many purposes, including navigation, irrigation, drinking water supply, and recreation. At the same time, it poses a persistent threat. The country must be protected against flooding, as well as the wider consequences of water-related disturbances [Van der Brugge and De Winter, 2024].
Responsibility for water safety in the Netherlands lies with the Minister of Infrastructure and Water Management (I&W). Protection is ensured through an extensive system of dikes and river diversions, which has made the Netherlands a leading international example of flood management [Algemene Rekenkamer, 2023].
For decades, the Dutch flood protection strategy relied on defending against extreme discharges defined by exceedance probabilities. Primary flood defences were reinforced accordingly, designed to withstand fixed water levels [Slootjes and Van der Most, 2016].

Following the North Sea flood disaster of 1953, a large-scale reinforcement programme was introduced. This programme was guided by an economic optimum, balancing the costs of flood protection against the damages avoided, and required the strengthening of many levee segments [Mosselman, 2022]. The public broadly accepted the initial reinforcements. Subsequent reinforcements, however, were needed relatively soon after due to new technical insights and encountered increasing resistance [Mosselman, 2022].
During this new wave of reinforcements, local stakeholders were often overlooked, and houses located on the outer side of the dike were demolished. This approach provoked strong protests against the large-scale dike designs. In response to public pressure, the government lowered the exceedance probability of dikes to 1 in 1,250 per year, which allowed for slimmer dikes [Van Heezik, 2006].

Nevertheless, dikes remained oversized. Instead of adopting a feasible alternative strategy of lowering floodwater levels by giving the river more space [Mosselman, 2022], the Boertien Commission proposed a new statistical distribution in 1993. This revision enabled lower design discharges at the same exceedance probability, resulting in substantial reductions in dike height [Van Heezik, 2006, Jonkman et al., 2021].
The floods of 1993 and 1995, however, reversed this trend. Updated statistics showed that the dikes no longer met safety standards. In addition, many reinforcements were not yet completed in 1993, which forced the evacuation of residents and altered public opinion on flood protection [Mosselman, 2022].

Rather than once more reverting to larger dike designs, the government eventually introduced the 'Room for the River' policy [Van Heezik, 2006]. This policy later evolved into the Beleidslijn Grote Rivieren (Policy Guideline for the Major Rivers, BGR) in 2006 [Rijkswaterstaat, Ministerie van Infrastructuur en Waterstaat, 2019].
The BGR seeks to safeguard river discharge capacity by restricting non-river-related construction in floodplains and by preventing obstructions that could limit future 'Room for the River' measures. Interventions are evaluated using the Rivierkundig Beoordelingskader voor de Grote Rivieren (River Assessment Framework for Interventions in Major Rivers, RBK) [Rijkswaterstaat, Ministerie van Infrastructuur en Waterstaat, 2019].
According to this framework, no intervention may result in adverse hydraulic effects.

A central criterion is the so-called one-millimetre rule: any increase in water level along the river axis caused by a flood defence intervention must not exceed one millimetre, unless fully compensated within the project. This threshold functions as a buffer for model uncertainty [Van Doorn and Rijkswaterstaat Water, Verkeer en Leefomgeving, 2023]. Further details on the BGR and RBK are provided in Appendices B.1 and B.2.

The maximum allowable water level increase of one millimetre is highly restrictive. This is particularly the case given the inherent uncertainties in the hydrological models used to evaluate river interventions. These models contain several sources of uncertainty, including model, stochastic, and forecasting uncertainty. Calibration at the millimetre level may suggest a level of accuracy that cannot realistically be achieved. Such detailed calibration risks overstating the reliability of the model [Berends and Diermanse, 2021], even though both the baseline and intervention scenarios are subject to similar uncertainties.
For instance, the uncertainty in design water levels for the River Waal caused by bed forms and vegetation roughness, exceeds the one-millimetre threshold by more than a factor of one hundred [Warmink et al., 2013]. Comparisons between modelled water levels may be more reliable than the absolute levels themselves. Yet, enforcing a one-millimetre limit attributes undue confidence to model results and neglects the uncertainty range these inevitably contain [Berends and Diermanse, 2021].

Due to the strict criteria set by the RBK, engineers have often avoided outward dike reinforcement as a design alternative [Kok et al., 2016]. Such reinforcements can cause adverse hydraulic effects (see Appendix A), particularly increases in water levels that must be mitigated to comply with the one-millimetre rule. Even minor outward expansions can generate significant additional costs for maintaining the same reference design water level. As a result, riverward reinforcement is frequently dismissed, as it can turn an otherwise straightforward dike project into a complex and costly endeavour due to the required mitigation measures (M. van den Berg, personal communication, February 2025).

1.2. New flood defence policy and the Flood Protection Programme

Predicted climate change is expected to increase annual high flows and extreme flood discharges in the Rhine catchment [Nilson et al., 2024]. At the same time, projected socio-economic growth in the Netherlands, reflected in the ’Vlug ’24’ and ’Ruim ’24’ scenarios of the Dutch Delta Programme [Van der Brugge and De Winter, 2024], reduces the space available for flood defences. As a result, pressure on the water system is increasing. For all four climate scenarios developed by the IPCC, water-related challenges are expected to intensify for the Netherlands [Van der Brugge and De Winter, 2024].
In 2017, the Minister of I&W introduced a new policy for national water safety. As part of this policy, a new approach to flood safety was introduced, in which flood defences are designed based on the probability of flooding. Since a breach does not necessarily result in flooding, the probability of flooding is defined as “the probability of the loss of flood defence capacity in a levee segment causing the area protected by the levee segment to flood in such a way that fatalities or substantial economic damage occur” [Kok et al., 2016, p. 64].

The study Veiligheid Nederland in Kaart (VNK2) determined the maximum allowable failure probabilities for each dike segment. This assessment was based on the maximum permissible local individual risk (LIR) of flooding, flood simulations, and a social cost-benefit analysis [Slootjes and Van der Most, 2016]. The LIR is defined as “the probability of an individual dying somewhere as a result of flooding, which may not exceed 1/100,000 per year” [Kok et al., 2016, p. 51].

To meet the newly defined safety standard, the entire Dutch dike system has been reassessed. If levee segments do not comply with the required maximum probability of flooding, the flood defences must be reinforced by 2050 [Hoogwaterbeschermingsprogramma, 2024a]. Based on the assessment, approximately 1500 km of dikes, which is 63% of the country’s primary flood defences, require upgrades by this deadline [Hoogwaterbeschermingsprogramma, 2024a]. To facilitate this process, the minister established the ’Hoogwaterbeschermingsprogramma’ (HWBP, Flood Protection Programme), which is managed by Rijkswaterstaat (RWS) and the water boards. However, initial evaluations suggest that achieving all necessary upgrades before 2050 may not be feasible.

1.3. Problem statement

A challenge is that dike reinforcement projects are progressing more slowly than anticipated. Additionally, the costs of these improvements have proven to be higher than expected, leading to financial constraints [Hoogwaterbeschermingsprogramma, 2024a]. There is an ongoing debate about whether the 2050 deadline is too ambitious for completing all primary flood defence upgrades [Algemene Rekenkamer, 2023, Hoogwaterbeschermingsprogramma, 2024a].

The limited availability of space presents a significant constraint on dike reinforcement efforts. In the Netherlands, increasing land-use pressure has reduced support for dike improvements that require inward expansion [Algemene Rekenkamer, 2023, Wichman, n.d.]. Local residents prefer visible reinforcement projects that minimise spatial impact and preserve views of the waterline [Algemene Rekenkamer, 2023], while avoiding overly intrusive designs, which have faced opposition in the past [Van Heezik, 2006]. A case study of HWBP dike reinforcement projects (which is discussed in more detail in Appendix C) shows that inward reinforcement was often preferred from a design perspective, but rarely implemented due to the need to preserve monuments, buildings, tree lines, scenic landscapes, and social cohesion.

As a result, riverside (outward) reinforcement is generally preferred [Algemene Rekenkamer, 2023]. However, as discussed in Section 1.1, it is often avoided because of strict regulations and the additional complexity it creates.
The Flood Protection Programme has developed a rationale for outward reinforcement (Appendix B.3). This rationale sets out the criteria under which outward expansion may be applied when inward expansion is not feasible or too costly, allowing a soil body to be constructed on the riverside instead of relying on sheet piles. Nevertheless, the RBK's criteria still apply, which increases project complexity, scale, and cost [Hoogwaterbeschermingsprogramma, 2024e].

The findings from the case study show that only dikes located in the lee of the flow or at a greater distance from the riverbed were deemed suitable for outward expansion. In most other cases, even a water level increase of a few millimetres was sufficient to rule out this option.
Outward expansion is also often deprioritised in favour of preserving a continuous dike trajectory and maintaining flow alignment. In addition, landscape and scenic qualities, historical values, and the protection of existing buildings are frequently prioritised over purely cost-optimal or technically efficient solutions.

Given the constraints of limited space, stakeholder concerns, and strict regulations, soil-based expansions are often not feasible. In such cases, flood defence designs frequently rely on sheet piles. These structures are compact and can be more practical where space is scarce, particularly if the costs of acquiring land for wider soil dikes or implementing special measures become prohibitive [Algemene Rekenkamer, 2023, Stuurgroep Meanderende Maas, 2023a].
Although sheet piles may be economical when space is limited, the structures are inherently costly to implement. Additionally, the adaptability of sheet piles is limited. Once the technical lifespan ends or safety standards change, flood defences incorporating sheet piles often require complete replacement. Unlike soil-based dikes, the old structures are difficult to remove and therefore fixate the alignment of the dike (P. van der Scheer, personal communication, May 2025).

Need for adaptability

Evidence increasingly suggests that sea level rise could exceed the 0.3–1 metre range by 2100, which was used as the basis for the Kennisprogramma Zeespiegelstijging (Sea Level Rise Knowledge Programme, KS ZSS). Rising sea levels would also elevate river water levels and extend their influence further inland [Haasnoot and Diermanse, 2022].

KS ZSS has proposed four adaptive pathways to address sea level rise, but no definitive path has yet been selected. The programme therefore strongly recommends identifying low-regret solutions that are robust and flexible, as these are essential to enable future adjustments. The coming decade is considered critical for the Netherlands because of the urgency of climate mitigation and the limited availability of space. Agricultural and housing developments further constrain the options for adaptation [Haasnoot and Diermanse, 2022].

Strict hydraulic criteria and stakeholder preferences often lead to the selection of sheet piles. This restricts adaptability by fixing the dike trajectory, thus complicating potential future adjustments. Soil-based dikes offer greater flexibility and reuse potential, but are currently limited by societal values and water level regulations. The tension between short-term feasibility and long-term adaptability underscores the need for reinforcement alternatives that are both technically feasible and represent robust, low-regret solutions. Accordingly, alternatives such as riverward dike reinforcement under relaxed regulatory conditions warrant further exploration.

1.4. Research objective and questions

The goal of this thesis is to determine, through the use of conceptual designs, whether variants of outward dike reinforcement (ODR) can serve as technically feasible and cost-effective alternatives to inward or construction-based reinforcement strategies, when assessed under relaxed regulatory conditions. This evaluation is carried out under a relaxed interpretation of the Rivierkundig Beoordelingskader (RBK), in which water level increases caused by ODR are assumed to be absorbed by the flood defences themselves, without requiring additional mitigation measures such as floodplain lowering or expansion. The results aim to support engineering practice by providing insights and first-order design graphs into the applicability of ODR in contexts where strict regulatory constraints would not be enforced. Furthermore, this thesis tries to identify elements within the RBK that may warrant additional evaluation or further relaxation to facilitate the use of ODR as a primary flood defence.

In this thesis, (technical) feasibility refers to the ability of affected dikes to maintain acceptable performance under altered hydraulic conditions induced by ODR, throughout their designed functional lifetime. This assessment is based on the primary failure mechanisms of a dike, and if performance is compromised, mitigation strategies are considered. The term does not refer to the feasibility of the riverward dike itself, since such variants can be designed to perform as desired, but rather to its influence on the surrounding flood defence system. Affected dikes are defined as all other dikes upstream or on the opposite bank that experience hydraulic impacts resulting from the implementation of the ODR alternative.

Although all RBK regulations are relaxed, this thesis primarily focuses on water-level effects, using the one-millimetre rule as a benchmark to interpret hydraulic outcomes. This rule provides a reference point for assessing the significance of water level changes induced by ODR. Further technical details and criteria of the RBK can be retrieved from Appendix B.

The following main research question guides this thesis:

To what extent is outward dike reinforcement under a relaxed regulatory framework of the 'Rivierkundig Beoordelingskader' a feasible and cost-effective primary flood defence?

The following sub questions will be addressed to develop a comprehensive understanding necessary to answer the main research question:

- Sub-question 1: What is the hydraulic impact of outward dike reinforcement, and which factors most significantly influence the resulting water level differences?
- Sub-question 2: To what extent is outward dike reinforcement technically feasible, considering its impact on the performance and functional lifetime of affected dikes, and how can these impacts be mitigated?
- Sub-question 3: How does outward dike reinforcement compare to non-outward alternatives in cost-effectiveness, both within a single reinforcement cycle and over a longer forecast horizon?

1.5. Design-based approach and hypothetical case framing

This thesis adopts a research-through-design approach, in which conceptual dike reinforcement variants are developed to investigate the hydraulic impact, technical feasibility, and cost-effectiveness of ODR. These designs are not intended as final engineering solutions, but are created as an analytical tool to frame results within a realistic context. This method is particularly useful for research into dike reinforcement, where uncertainty, stakeholder diversity, and interdependent design choices create a complex decision-making environment. By framing the research through conceptual designs, it becomes possible to explore these complexities in a structured and tractable way. As articulated by Stappers and Giaccardi, this approach "shows a hitherto nonexistent combination of factors as a provocation for discussion" and "in the making that brought this prototype into existence, the designer(s) will have struggled with opportunities and constraints, with implications of theoretical goals/constructs, and the confrontation between these and the empirical realities in the world." [2014, Section 41.1.4].
Although strictly research-through-design requires the discussion and elaboration of prototypes, intermediate steps and final decisions [Stappers and Giaccardi, 2014], this will not be addressed in this thesis. Only the final dike designs are used for the subsequent assessments relevant to the research questions.

To ensure realism in the conceptual designs and subsequent analyses, this thesis is framed within a hypothetical reinforcement of the southern Waal bank under the Flood Protection Programme, specifically between the reach of Dreumel and Nijmegen. The case area is presented in Figure 1.1. This provides a concrete context for hydraulic conditions, river dimensions, safety norms, soil characteristics, and projected installation dates. Furthermore, the RBK has dedicated its own section to the Waal River, which allows a clear overview and discussion of the relaxed regulations. To maintain a clear technical scope, ecological considerations, nature values, and stakeholder interests are excluded from this thesis.

Figure 1.1: Visualisation of the hypothetical case area considered in this thesis. Image retrieved from Google Earth.

Moreover, the design phases of the northern Waal bank reinforcements provide a valuable source of reference designs, design insights and stakeholder perspectives that further inform this thesis. The reinforcement projects, covering approximately 70 kilometres from Gorinchem to Nijmegen, were finalised before 2025 as part of the Flood Protection Programme. While some sections remain under construction, the availability of detailed project documentation enables a retrospective case study that identifies design choices and stakeholder considerations. This case study is presented in Appendix C, and these insights, along with the specific assumptions and design considerations of the conceptual dikes, are elaborated in Chapter 2.

1.6. Modelling scope, conservative assumptions and stress scenarios

The modelling scope of this thesis is limited to identifying relative trends between conceptual dike variants, rather than producing execution-level precision. This is appropriate given the hypothetical nature of the reinforcement project. Accordingly, the analytical chapters use deterministic and simplified formulations in combination with expert heuristics to assess the hydraulic, technical, and economic implications of outward dike expansion, which is in line with the preliminary design level [Kok et al., 2016]. Only Chapter 4 includes a more detailed hydraulic assessment using the numerical software D-Hydro Suite.

To ensure robustness, this thesis consistently applies conservative assumptions when uncertainty exists in design choices or when selecting input variables. This conservative framing ensures that the conclusions remain valid even under extreme but plausible conditions, suggesting that the real-world applicability of ODR may be more favourable than the findings presented in this thesis indicate. Stress-test conditions are used to frame the analysis, including the maximum feasible discharge scenario in the Waal and the most conservative associated hydraulic and climate conditions.

1.7. Thesis structure, outline and conceptual framework

This section outlines the structure of the thesis and clarifies the decentralised positioning of methods. A central methodology chapter is deliberately omitted, as each analytical chapter addresses a distinct theme requiring a tailored approach. By embedding the methodology within each chapter, the relevance and readability of the analyses are improved. To ensure consistency and comparability, each analytical chapter follows a similar structure: a concise explanation of the approach and methods, results for the Waal River including a sensitivity analysis, and a concluding reflection. This format allows chapters to be read independently, while maintaining coherence through their shared focus on the implications of ODR.

To provide the input for the analytical chapters, this thesis first introduces four conceptual reinforcement variants and assesses their adaptability. **Chapter 2** presents four conceptual variants: an inward-based design, a construction-based design, and two ODR variants. These ODR variants reflect the maximum and minimum extent of outward expansion, thereby delineating the range of hydraulic impact and the scope of interpretable results. Additionally, adaptability of these conceptual dikes is assessed, which later plays a role in the cost-effectiveness analysis.

The three analytical chapters: hydraulic impact assessment (Chapter 3), feasibility assessment based on affected dike performance (Chapter 4), and cost-effectiveness assessment (Chapter 5) form the core of this thesis. These rely directly on the conceptual designs and are methodologically linked, with each chapter building on the results of the previous one to form a coherent assessment of ODR, as shown in the conceptual framework in Figure 1.2.

Chapter 3 assesses the hydraulic impact of outward dike expansion, which informs the subsequent feasibility and cost-effectiveness assessments. A simplified 1D compound channel model is used to explore general sensitivity patterns and hydraulic trends. This is complemented by a detailed simulation using the numerical D-Hydro Suite model, which provides site-specific insights for the Waal River that cannot be captured by the simplified model. Together, these analyses define both the expected range of water level differences and the precise local impact. This forms the basis for evaluating technical feasibility and mitigation strategies, and provides first-order design insights relevant to engineering practice. This chapter answers the first sub-question.

Chapter 4 evaluates the technical feasibility of outward dike expansion by assessing whether affected dikes maintain acceptable performance under the altered hydraulic conditions throughout their design lifetime. The analysis focuses on the main failure mechanisms and identifies limitations to ODR where safety norms are not met. In such cases, functional lifetime reduction (FLR) is determined, and two mitigation strategies are proposed. These serve as input for the cost-effectiveness comparison in Chapter 5. This chapter answers the second sub-question

Chapter 5 compares the short- and long-term costs of the conceptual designs under varying reinforcement strategies and future scenarios, to identify under which conditions ODR offers the most cost-effective solution. In the short term, a single reinforcement round is assessed to highlight relative cost differences between the conceptual designs. Subsequently, the cost assessment over a longer forecast horizon is based on life cycle cost analysis (LCC), using net present value (NPV) to evaluate each alternative over time. This analysis builds on the feasibility outcomes of Chapter 4 and compares two mitigation strategies to assess economic viability across different planning horizons. This chapter answers the final sub-question.

Following the analytical chapters, **Chapter 6** reflects on the validity and limitations of the approach, designs, and findings, and discusses the implications of ODR implementation for long-term planning and adaptive reinforcement strategies. **Chapter 7** summarises the findings of chapters 4 to 6 and answers the research questions. Lastly, **Chapter 8** provides recommendations for policymakers, practitioners and future research.

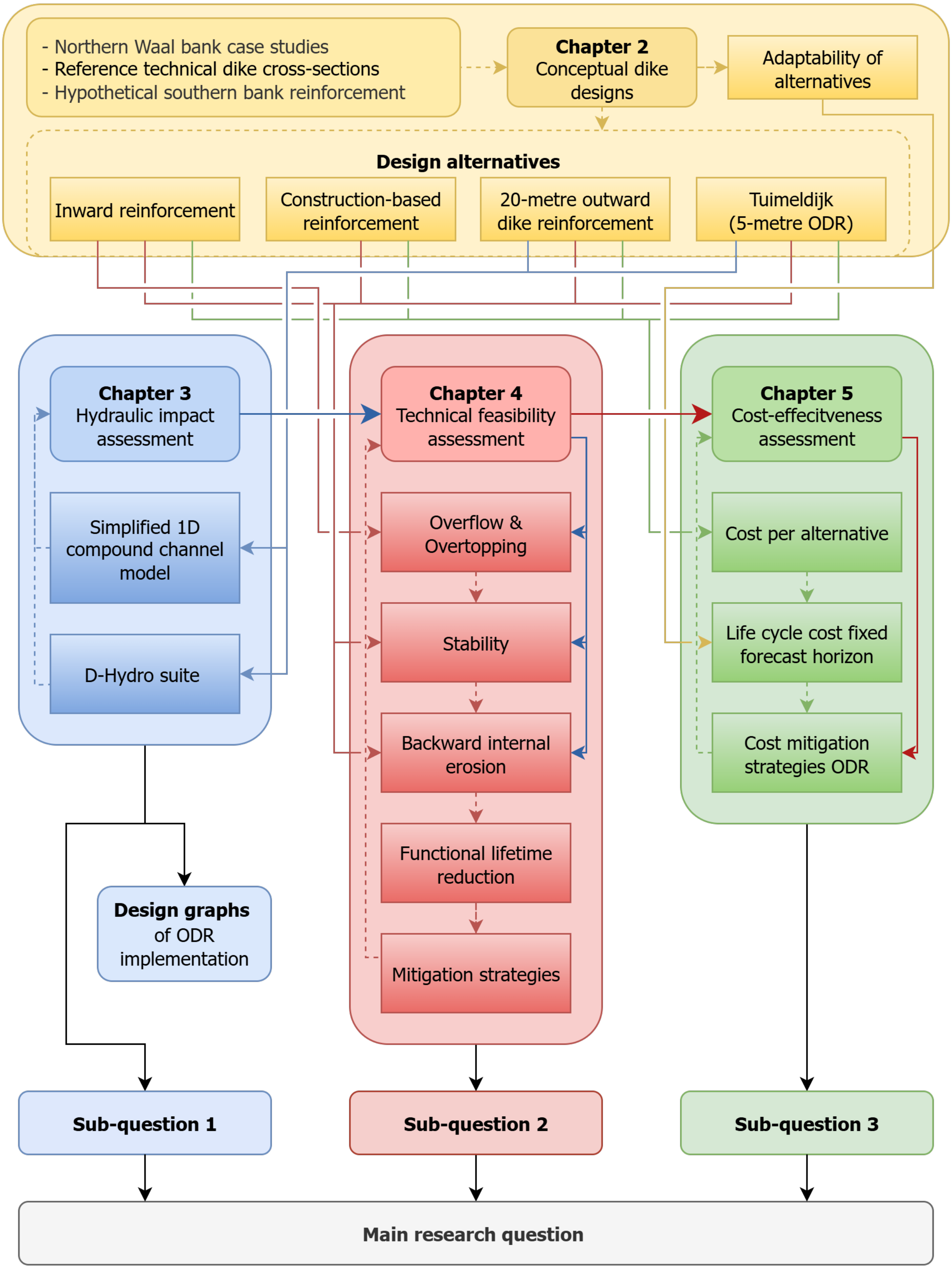

Figure 1.2: Conceptual framework of this thesis. The yellow box (Chapter 2) shows the development of conceptual dike designs, which serve as input for the analytical assessments. The blue box (Chapter 3) represents the hydraulic impact analysis, which directly informs the technical feasibility assessment in Chapter 4 (red box). The resulting functional lifetime reduction affects the cost of ODR, linking to the cost-effectiveness assessment in Chapter 5 (green box). The adaptability of the conceptual designs, introduced in Chapter 2, is also applied in the cost analysis.

2

Conceptual dike designs

This chapter introduces four conceptual dike designs developed at the preliminary design level, providing a realistic framework for the analytical chapters. First, the design basis is elaborated, drawing on reference designs, expert heuristics, key failure mechanisms, and a case study from previous reinforcements along the northern bank of the Waal. Next, the four conceptual dike variants are presented. Finally, their adaptability is briefly assessed based on a defined adaptability criterion.

2.1. Basis for conceptual dike designs

Reference dike designs

The four conceptual dike designs are based on two approved reference profiles from the detailed design phase of the Neder-Betuwe reinforcement project, to ensure practical relevance. The Neder-Betuwe project forms part of the dike works on the northern banks of the Waal. These redesigned cross-sections, DT095 and DT104, are located near Ochten and represent distinct approaches: a construction-oriented design and a soil-based design with mild outward expansion (Figures 2.1 and 2.2) [Waterschap Rivierenland and Royal HaskoningDHV, 2022]. As the locations are in close proximity, similar soil and hydraulic conditions are expected, and the same hydraulic load level is assumed. Both cross-sections have been approved in practice, indicating no expected susceptibility to backward internal erosion or stability failure. For the conceptual dike designs, additional assumptions are required regarding berm effects and construction detailing.

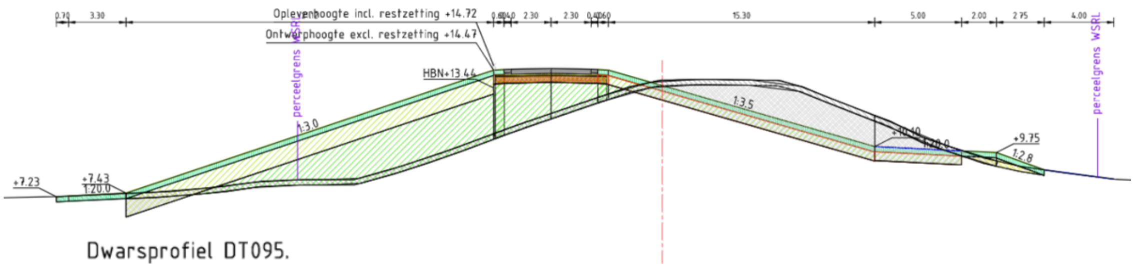

Figure 2.1: Technical cross-section of dike design DT095 of the Neder-Betuwe project. [Waterschap Rivierenland and Royal HaskoningDHV, 2022]

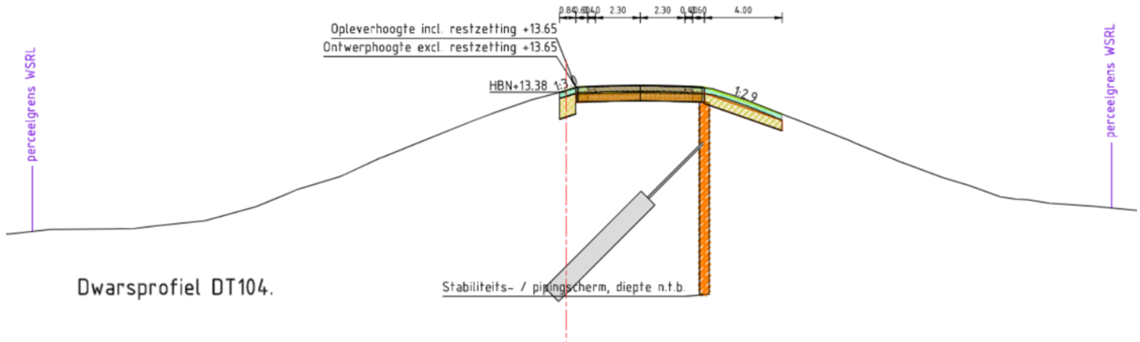

Figure 2.2: Technical cross-section of dike design DT104 in the Neder-Betuwe project. [Waterschap Rivierenland and Royal HaskoningDHV, 2022]

Besides the new cross-sections, the previously situated dike that required reinforcement is also shown in Figure 2.1. Based on this, the following assumptions are made:

- The elevation level of the hinterland is taken from the reference cross-section, measured at 7.79 m +NAP at the location of the old inner toe of the dike.
- The elevation level of the outer toe of the old dike is assumed to be at 7.00 m +NAP.
- The soil in the area where the old dike was situated is assumed to have been preloaded.
- Potential cables or pipelines near the dike toes are not taken into consideration.

Dimensioning based on failure mechanisms

The conceptual dike designs are developed in compliance with technical safety standards, flood protection performance, and current engineering regulations associated with the Waal River. Dimensioning is based on the three primary failure mechanisms: overtopping, slope stability, and backward internal erosion [Van Mierlo et al., 2007, Groenewoud, 2016]. The modelling approach and conservative assumptions underlying this thesis are described in the introduction.

For the dimensions associated with the overtopping and internal erosion criteria, deterministic exceedance-based approaches are applied, using the maximum design discharge of the Rhine at Lobith, which is 18,000 m^3/s [De Vriend et al., 2017]. These methods are further elaborated in Chapter 4 and Appendix E.

Stability is primarily assessed using expert judgement and practical engineering heuristics, focusing on the presence of a 20-metre berm or sufficiently gentle inner slopes to ensure macro-stability during high water. According to expert input (C. Spoorenberg, personal communication, May 2025), a berm of up to 20 metres is considered sufficient for stability if space allows. If no berm can be constructed, stability is evaluated using slope-based heuristics: an average inner slope shallower than 1:4 is generally acceptable [Jonkman et al., 2021], while steeper slopes increase the risk of instability ['t Hart et al., 2016, Boskalis and Royal HaskoningDHV, 2022], though a 1:3 slope may still be justified after detailed design (C. Spoorenberg, personal communication, May 2025).

If stability cannot be confirmed through expert judgement, a simplified geometric assessment is applied to ensure a consistent and practical evaluation. A dike is considered stable if one of the following conditions is met:

- An inner slope of approximately 1 : 4 (including berms and crest surplus) is present from the inner crest to the inner toe, and an outer slope of 1 : 3 is achieved.
- The cross-section corresponds to one of the two reference dike designs near Ochten, for which stability has already been verified.
- If a stability berm is present, it must start at one-third of the crest–hinterland height difference and have a 1 : 20 slope for drainage. This configuration is considered stable based on expert judgement.

If none of the stability conditions are met, it is assumed that steel sheet piles are required. A common rule of thumb is that the sheet pile length should be three times the height difference between the crest and the hinterland, with one-third embedded in the crest and two-thirds in the subsoil. If the subsoil consists of weaker Holocene sand, this full length is necessary. However, if stiff Pleistocene sand is present, the soil provides sufficient strength, although buckling becomes a concern. In such cases, the sheet piles are limited to 5 m into the Pleistocene layer and must be anchored with grouted ties (C. Spoorenberg, personal communication, May 2025).

Structural reinforcement for the backward internal erosion mechanism is considered if both the piping and heave sub-criteria indicate failure, with both the assessment and required sheet pile length determined using Sellmeijer's formulations. Given the spatial constraints assumed in this study, soil-based solutions are excluded and mitigation is addressed through the use of sheet piles. Steel sheet piles are assumed to be installed at the inner toe of the dike where the exit point is most likely to occur, provided no ditch or crack is present in the blanket layer [Förster et al., 2012]. If stability issues are also identified, the sheet pile is placed at the crest. In such cases, the required length is governed by the more critical of the two mechanisms: internal erosion or slope stability. Sheet pile lengths are rounded up to half-metre increments, as is common practice in dike reinforcement projects by contractors (P. van der Scheer, personal communication, April 2025). Further calculation details are provided in Appendix E.3.1.

Dike segments are defined by their location and geometry, with local subsoil conditions expected to influence the response of internal erosion mechanisms to outward dike expansion. To maintain focus, this study evaluates simplified dike cross-sections with similar subsoil conditions. Soil parameters are only varied in the sensitivity analysis.

The assumptions regarding internal erosion reflect a conservative approach. However, such conservatism is also applied in practice during preliminary investigations, especially when soil properties have not yet been fully verified (C. Spoorenberg, personal communication, May 2025).

Design choices typical for the Waal area

In addition to the deterministic approach described previously, this section outlines design choices and assumptions informed by completed reinforcement projects along the northern Waal banks (see Appendix C), ensuring that the conceptual designs reflect common practice and typical local conditions along the Waal River. The main assumptions are outlined below:

- Outward slopes of 1 : 3 are applied to all variants. This slope is commonly used due to its visual aesthetics and is imposed as the maximum steepness to ensure accessibility for maintenance and operational activities (C. Spoorenberg, personal communication, May 2025).
- The overtopping discharge is set at 10 l/s per metre.
- Each dike body includes a 0.2-metre top layer of potting soil (grass cover) for erosion and overtopping protection, followed by a 0.8-metre impermeable clay layer of C2 quality (A. den Teuling, personal communication, May 2025).
- The remainder of the dike core is filled with sand. Approximately 80% of the original sand core remains unaffected during reinforcement, provided that the dike does not need to be relocated. This approach is commonly applied in practice (A. den Teuling, personal communication, May 2025).
- All dike crests feature a 6-metre-wide road with additional edge closure and foundation, as has been standard practice in reinforcement projects along the northern banks of the Waal (Appendix C).
- In line with the hydraulic load level (HBN) for the reference cross-section DT095 at Ochten, the HBN of the conceptual dike designs is set to 13.44 m +NAP. However, for the sheet pile calculations, this is set to 12.50 m +NAP, as discussed in Appendix E.3.1.
- For these designs, a conservative assumption is made that the seepage length spans from the inner toe to the outer toe of the dike.

Additionally, dikes require a surplus height to account for settlement, consolidation, and autonomous subsidence (C. Spoorenberg, personal communication, July 2025). The following assumptions are applied:

- The hydraulic load level is expected to rise by approximately 30 cm over a 50-year functional lifetime, or 50 cm over 100 years (see Chapter 4). This is not added to the surplus height, but results in additional settlement.
- An additional 15 cm is assumed for settlement.
- Autonomous subsidence in the Waal region is estimated at 1 mm/year [SkyGeo, 2023], corresponding to 5 cm over 50 years.
- For consolidation, 10% of the new height is considered sufficient, resulting in approximately 5 cm.
- This leads to a required surplus height of 25 cm, corresponding to a linear subsidence rate of 5 mm/year.
- If the subsoil is not preloaded, a surplus height of 50 cm is assumed, with a linear subsidence rate of 1 cm/year.
- This surplus height is applied to both soil-based and construction-based designs. Although construction-based designs may result in higher HBN values and thus greater consolidation, it is assumed that significant preloading has already occurred to account for settlement.

2.2. Presentation of the four conceptual dike designs

2.2.1. Inward dike reinforcement

The conceptual dike design of the inward reinforced is shown in Figure 2.3. In this design, it is assumed that an additional 6 metres of inward expansion is required, as if the space adjacent to the inner toe of the old dike is not yet available.

The crest height of the original dike was 13.90 m +NAP, which is retained in the new design. Lowering the crest would be illogical, especially given the assumption that land acquisition on the landward side will be necessary in any case, regardless of the specific crest height.

The dike slopes comply with the stability assumptions outlined in Section 2.1. Based on these geometric criteria, and in the absence of indications of instability, no sheet piles are considered necessary to ensure structural stability.

Additionally, the design results in a seepage length of 55.4 metres from toe to toe. For typical soil conditions along the Waal, based on the parameters described in and calculated in Appendix E.3.1, this length is sufficient to satisfy the heave criterion. Consequently, no sheet piles are required to mitigate the backward internal erosion failure mechanism under standard conditions.
However, in areas with weaker soil profiles, backward internal erosion may become critical. In such cases, sheet piles may be necessary to meet the heave criterion, which would substantially increase the cost of this alternative. The methodology used to assess the need for sheet piles, as well as the sensitivity analysis for scenarios involving weaker soil conditions, is further detailed in Appendix E.3.

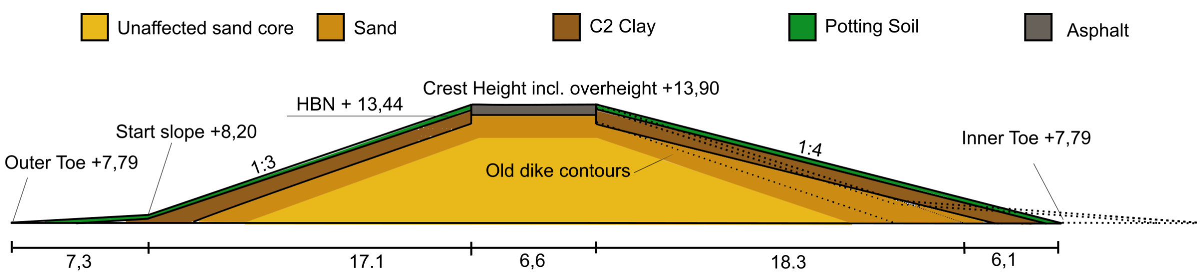

Figure 2.3: Conceptual design alternative of inward reinforcement

2.2.2. Construction-based dike reinforcement

The second conceptual dike design represents a construction-based reinforcement, as illustrated in Figure 2.4. This design occupies the least spatial footprint and is intended for situations where no additional space is available on either side of the existing dike. Consequently, the reinforced structure is even smaller than the original dike (indicated by the dotted contours), reflecting the spatial constraints characterising this design.

Although a construction-based design typically has a functional lifetime of 100 years, and would therefore logically be associated with a higher HBN, this was not the case for reference dike DT104. Consequently, the HBN is not adjusted in this study and remains consistent with the other designs.

While the inner slope of the dike does not meet the geometric stability criteria outlined in Section 2.1, stability is ensured by implementing sheet piles. This decision is supported by the similarity to cross-section DT104 near Ochten, where sheet piles are required to maintain structural integrity.
The subsurface beneath the dike primarily consists of Pleistocene sand, overlain by a thin blanket layer and a thin Holocene sand layer, as described in Appendix E. If the sheet pile were to extend three times the height difference between the crest and the hinterland level, buckling could occur due to the relatively stiff Pleistocene sand. To prevent this, the sheet pile length is limited to 5 metres below the blanket layer, penetrating the Pleistocene sand. This results in a total, rounded-up steel sheet pile length of 13.5 metres at the crest, supplemented with grouted anchors to ensure stability.

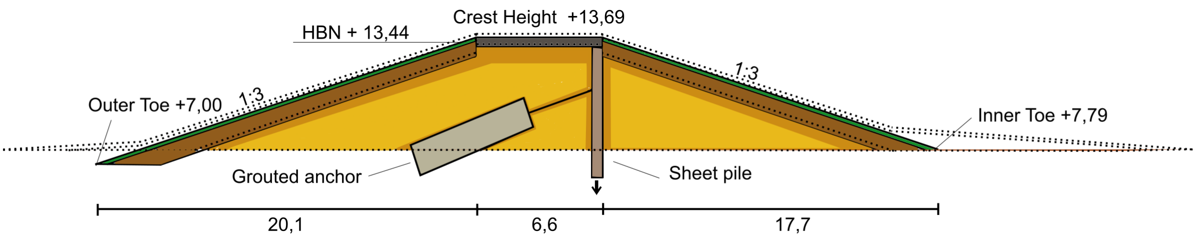

Figure 2.4: Conceptual design alternative of construction-based reinforcement

2.2.3. 20-Metre outward dike reinforcement (20-metre ODR)

The third conceptual design represents the maximum extent of outward dike reinforcement considered in this thesis, featuring a 20-metre expansion as illustrated in Figure 2.5. Based on the extent of outward expansion, this variant is expected to have the largest hydraulic, technical, and economic impacts among the conceptual designs. This expectation will be evaluated in the subsequent analytical chapters.

A 20-metre outward expansion is considered the maximum necessary to ensure stability, as previously justified in Section 2.1. This width also provides sufficient resistance against backward internal erosion, eliminating the need for sheet piles. Although greater expansions would further improve performance, particularly for the heave criterion, these offer limited added value in light of spatial constraints in reinforcement projects considered in this study.

Although the actual design extends approximately 18 metres beyond the reference, this study adopts 20 metres as the maximum input variable for the hydraulic impact analysis. This provides flexibility in the design process and greater adaptability in future implementation, as discussed in more detail in Section 2.3.

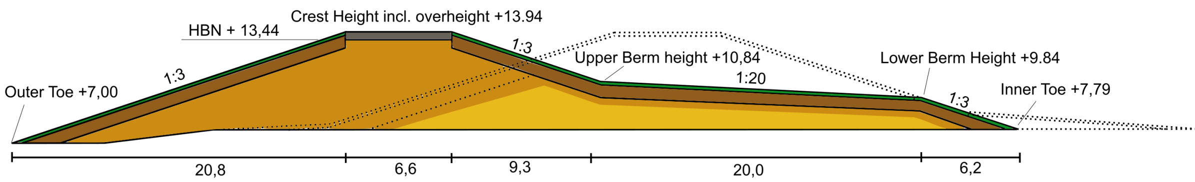

Figure 2.5: Conceptual design alternative of outward dike reinforcement

2.2.4. Tuimeldijk (5-metre ODR)

The fourth and final alternative, known as a Tuimeldijk, features a 5-metre outward reinforcement in which an additional structure is constructed directly against the existing dike. The new crest is slightly elevated and includes a cycle or footpath, as applied in the Meanderende Maas project [Stuurgroep Meanderende Maas, 2023b]. The design used in this thesis is shown in Figure 2.6. This variant demonstrates that outward reinforcement does not necessarily require a large berm, although its applicability may be limited by local conditions, as discussed in Appendix E.3.1. As the minimum outward expansion considered in this thesis, the Tuimeldijk is expected to have the lowest hydraulic, technical, and economic impacts among the outward reinforcement variants.

Although the original dike crest is at 13.90 m +NAP, the Tuimeldijk crest is raised by 50 cm to improve traffic safety and aesthetics by clearly separating the bike path from the road, as also applied in the Meanderende Maas project [Waterschap Aa en Maas et al., 2022].

To ensure stability, the slope in between the two crests is set to 1 : 5, resulting in an overall inner slope of approximately 1 : 4, which is considered sufficient. This assumes the inner toe lies along the extension of the original 1 : 3 slope, excluding any additional length behind it, to remain conservative.

Since the original dike remains intact and only a small crest is added, the inner toe is illustrated as preserved in Figure 2.6. However, to maintain a conservative estimate, and consistent with the approach used for other variants, this section is excluded from the seepage length calculation, as it lacks a substantial dike body.
Despite this exclusion, the total seepage length of 52.1 metres marginally satisfies the heave criterion, hence eliminating the need for sheet piles under the conditions analysed. This result is further substantiated in Appendix E.3.1. Nevertheless, weaker soil profiles may still pose a risk of backward internal erosion, similar to the inward design. In such cases, installation of sheet piles may be required.
Furthermore, the preserved inner toe is not considered available for future reinforcement. As a result, the design lacks flexibility, and any future adaptation would necessitate a completely new structural concept. These implications are further discussed in Section 2.3.

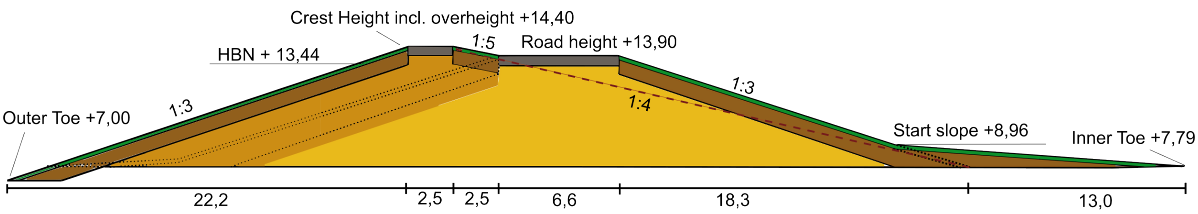

Figure 2.6: Conceptual design alternative of a Tuimeldijk

2.2.5. Conceptual designs overview table

Table 2.2 presents the key dimensions and structural characteristics of the four conceptual dike designs used in subsequent analyses. As these designs are conceptual and intended to provide indicative insights, they have not been optimised. Calculated volumes are rounded to whole numbers, and only the main dike cross-sections are included.

The volumes provided represent the required quantities for each new dike design. The required sand volume differs from the total sand volume, as part of the original dike core remains unaffected. This is particularly beneficial in the case of the Tuimeldijk. For the potting soil and clay layers, it is assumed that these materials need to be reapplied.

Table 2.1: Overview of the conceptual dike designs characteristics and volumes. All values are given per linear metre of dike cross-section.

	Inward reinforcement	Construction-based	Outward 20-metre	Tuimeldijk
Seepage length [m]	55.4	44.4	62.9	52.1
Land acquisition [m]	6, inward	–	20, outward	5, outward
Sheetpiles	no, possibly for heave	yes, 13.5 m incl. anchors	no	no, possibly for heave
Total sand volume [m^3/m]	128	120	157	167
Required new sand volume [m^3/m]	25	24	119	52
Required new clay volume [m^3/m]	31	27	42	18
Required new potting soil volume [m^3/m]	9	8	11	5

2.3. Adaptability of dike alternatives

Dike reinforcement involves long-term uncertainties, such as future hydraulic loads, spatial limitations, and evolving societal preferences. When reinforcement is not immediately urgent, adaptive designs offer the opportunity to anticipate future needs and reduce the risk of regret investments [Hoogwaterbeschermingsprogramma, 2023]

Implications of dike adaptability

Early consideration of subsequent reinforcement phases helps determine whether feasibility is retained under projected hydraulic loads and spatial constraints. If not, a more robust alternative may be preferable now, even if it requires a firmer spatial commitment. Reserving space today can reduce future costs and prevent legal or spatial conflicts, especially as land use pressure increases [Algemene Rekenkamer, 2023].

Adaptability is closely linked to uncertainty. Reserved space or conservative designs might ultimately prove unnecessary. Still, if these enable a soil-based solution, future modifications are simpler and more cost-effective than construction-based alternatives such as sheet piles. In this way, early spatial reservation lowers the risk of regret if conditions change.

Outward expansion is not inherently more desirable when buildings are present. However, if future reinforcement is anticipated, early acquisition or transparent planning can help avoid costly workarounds and societal resistance, leading to more low-regret decisions. Delayed reinforcement under spatial constraints may result in disruptive and complex upgrades. Early planning, therefore, supports both technical feasibility and smoother stakeholder engagement.

Adaptability of the four conceptual dike designs

The adaptability of the four conceptual dike designs is a key criterion for assessing their suitability as low-regret solutions in long-term flood risk management. Given the projected increase in hydraulic load, approximately 0.5 metres by 2085 and 1 metre by 2200 (see Figure 4.7, Section 4.2.2), it is essential to consider whether each design can accommodate future reinforcements within the available spatial envelope. In this study, a design is considered adaptable if it reserves sufficient horizontal space for future heightening, defined as seven times the additional crest height, with outer slopes of 1:3 and inner slopes of 1:4. While this criterion is not absolute, it provides a practical basis for comparison. Designs that meet this condition enable phased reinforcement and support transparent, long-term planning, reducing the risk of spatial or legal conflicts as land use pressure increases.

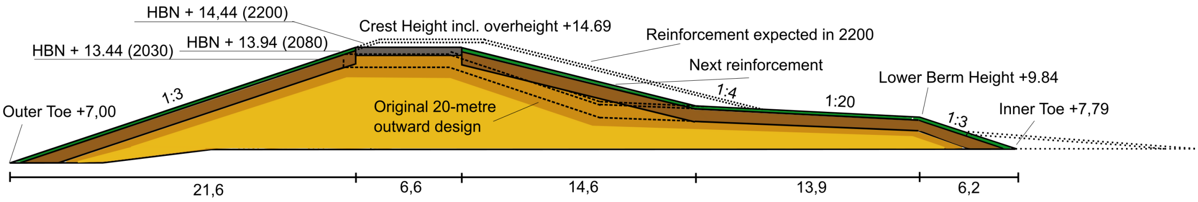

Figure 2.7: Adaptiveness of outward dike design; sufficient room for multiple reinforcement rounds.

Of the four designs, only the 20-metre outward reinforcement meets the adaptability criterion. The wide berm provides sufficient space for multiple future heightenings while maintaining overall stability (see Figure 2.7). Given the local soil conditions, backward erosion is not expected before 2200, so sheet piles are not required for subsequent reinforcement phases. Moreover, future reinforcements will require significantly less material, as the main investments in spatial reservation and earthworks are already made during the initial intervention, as illustrated in Table 2.2.

Table 2.2: Material requirements for the 20-metre outward dike design and its subsequent reinforcement, per linear metre dike.

	Outward 20-metre	Next reinforcement
Land acquisition [m]	20, outward	–
Total sand volume [m^3/m]	157	174
Required new sand volume [m^3/m]	119	48
Required new clay volume [m^3/m]	42	43
Required new potting soil volume [m^3/m]	11	12

The other three designs lack this adaptability:

- **Inward reinforcement**: Currently offers no additional space for future crest height increases. Further reinforcement would require new land purchases, either outward or inward, and therefore a complete redesign
- **Construction-based**: In addition to lacking space for future crest height increases, the sheet pile must be fully replaced once its functional lifetime ends and the old pile is hard to remove from the soil (P. van der Scheer, personal communication, April 2025). This makes the design potentially high-regret under future uncertainty.
- **Tuimeldijk**: Despite its additional crest height and expansion, it lacks reserved space for redesign. If future reinforcement is needed, additional stability measures or land acquisition would be required. The current configuration only just meets the stability and heave criteria, making it inflexible and potentially a low-regret option for now. Due to the expected hydraulic loads, sheet piles will be required for future reinforcements (see Appendix E.3.1).

In conclusion, only the 20-metre outward expansion truly offers adaptability. Because soil and space have already been secured, this design results in significantly lower costs for future reinforcements, as demonstrated later in this study. The other designs will require a complete redesign and new investments for each reinforcement phase. In particular, the Tuimeldijk will face substantially higher costs for its next reinforcement, since the use of sheet piles will be unavoidable.

3
Hydraulic impact assessment

3.1. Approach

This chapter evaluates the hydraulic impact of outward dike reinforcement (ODR) in the Waal River context, as a basis for evaluating its applicability in engineering practice. This chapter addresses the first subquestion. Two modelling approaches are applied: a simplified 1D compound channel model and the more advanced numerical D-Hydro suite model.

To gain insight into influential variables that dictate the hydraulic response of ODR, a simplified 1D compound channel model is applied, calibrated to the Waal River. This model enables a controlled sensitivity analysis and the identification of averaged hydraulic trends, and provides first-approximation graphs to support engineering practice in design contexts where strict hydraulic regulations may not apply.
The use of a compound channel is justified. Previous research has shown that for a river with a relatively constant width, such as the Waal, uncertainty propagation can be reliably assessed using a simplified cross-sectional approach. This enables a more controlled analysis of individual variables [Warmink et al., 2013]. The model is applied to the two outward variants, representing the expected range of hydraulic responses.

The D-Hydro suite simulations are applied to assess the detailed hydraulic effects of ODR in the Waal River, capturing local impacts that the simplified 1D model cannot resolve. Although D-Hydro is not an absolute representation of reality, it is widely used in Dutch practice to evaluate flood defence interventions in accordance with the Rivierkundig Beoordelingskader (RBK) [Van Doorn and Rijkswaterstaat Water, Verkeer en Leefomgeving, 2023]. In this study, it serves as a reference framework to translate the simplified model results into a more detailed and practice-oriented modelling environment, accounting for local variability and enabling verification of the 1D-model trends under realistic conditions.
The simulations explore variations in spatial application (within or outside bottlenecks), expansion magnitude and intervention length. The resulting quantified effects inform subsequent chapters and provide the most reliable estimates of hydraulic impact in the Waal River context.

The results section first examines the sensitivity of local floodplain characteristics, offering insight into the physical drivers that influence the hydraulic response to ODR. This is followed by analyses of spatial application, expansion magnitude and intervention length, aimed at identifying how and where ODR can be applied most effectively. These findings are then synthesised into generalised response graphs to support practical design decisions. The chapter concludes with a summary of the main insights, which inform the broader evaluation of ODR's applicability in engineering practice.

3.1.1. Model framework: Simplified 1D compound channel model

The simplified 1D hydrodynamic model applies the Chézy formula for uniform flow, in combination with the discharge equation for compound channels [Blom, 2025b], as shown in Equation 3.1. The compound channel consists of a deep and narrow compartment representing the main river channel (m) and a shallower, wider compartment representing the floodplain (f). The contribution of each compartment to the total discharge depends on its hydraulic roughness (Chézy constant C), width (B), and water depth (d), while the river slope (i_b) is constant across the cross-section. Note that the water depth in the channel compartment is defined as the sum of the water depth above the floodplain and the depth of the channel (d_c). The compound channel model is schematised in Figure 3.1.

$$Q_T = \left[\left(Cd^{3/2}B\right)_f + \left(Cd^{3/2}B\right)_m \right] \sqrt{i_b} \tag{3.1}$$

Outward dike reinforcement is modelled by expanding the outer width of the dike, effectively shifting the toe of the dike further into the floodplain, as illustrated in Figure 3.1. This reduces the available floodplain width and total conveyance area, causing the equilibrium water depth (d) to increase as a fixed discharge passes through a narrower cross-section.

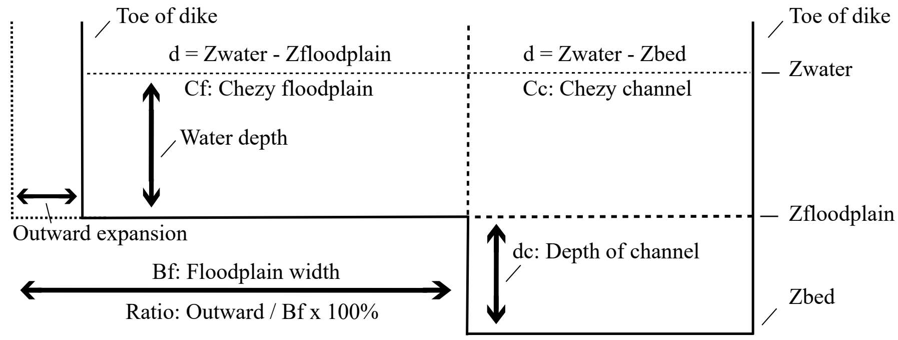

Figure 3.1: 1D cross-section schematisation of a simplified compound channel used in the hydrodynamic model. Outward expansion is modelled by shifting the boundary of the floodplain compartment into the floodplain (i.e. shifting the outer toe of the dike).

For the hydraulic impact analysis with the 1D model, the magnitude of floodplain width and outward expansion are varied, while the river slope, channel depth, and channel width are kept constant.
Although the model schematises only one floodplain, the results are considered representative of a configuration with floodplains on both sides. This is justified by the fact that for simplicity no hydraulic radius is used, and no additional turbulence due to crossover flow is incorporated to account for increased roughness. In reality, some flow may occur along the dike slope, but this is excluded to avoid introducing hydraulic radius calculations, which would add complexity for minimal gain in conveyance.

To enable generalised analyses of outward dike reinforcement, hydraulic roughness is deliberately simplified in this study. In reality, roughness in both the main channel and floodplain varies with discharge (i.e. water depth) and location, due to bed forms and vegetation [Domhof et al., 2018]. These variations contribute significantly to uncertainty in design water levels [Warmink et al., 2013]. In the 1D model, roughness is averaged per compartment and depth, and kept constant along the flow direction. While this simplification reduces the accuracy of water level predictions, it aligns with the scope of this study, as discussed in Chapter 1.

To avoid trivialising the effects of turbulence and local roughness variation, the 1D model is calibrated against D-Hydro Suite simulations of the Waal River, which account for these complexities. Calibration utilised multiple criteria, including the discharge–water level relationship and the simulated water level response to outward expansion. Although limited validation was feasible (discussed in Appendix F), these criteria suggested a reasonable, but slightly conservative, alignment between the simplified model and reference simulations. As a result, the 1D model reliably captures the general hydraulic behaviour of the Waal River in an averaged form, enabling controlled and generalised scenario analyses. Details of the calibration are provided in Appendix G.

Simulating backwater effects

Using the 1D model, the maximum water level difference (abbreviated as WLD) caused by outward dike expansion is calculated, representing the new equilibrium depth (d_e). In reality, this depth is not reached instantaneously along the river. The extent of the water level increase depends on the length of the outward reinforcement, referred to as the intervention length (Figure 3.2), which induces a backwater effect upstream. This upstream influence occurs because flow in the Waal River is subcritical [Berends et al., 2019]. A substantial distance is required before the flow gradually approaches the new equilibrium depth imposed by the expansion, reaching normal flow where gravitational and frictional forces are balanced.

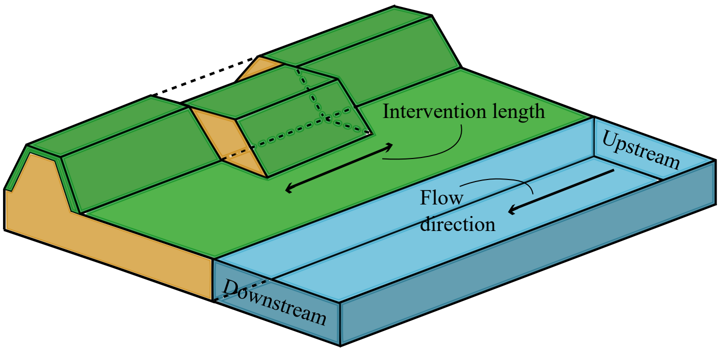

Figure 3.2: Intervention length of outward dike reinforcement visualised

Similarly, upstream of the expansion's starting point, the water level gradually transitions back towards its original equilibrium depth. The distance required for this spatial adjustment, referred to as the adaptation length and shown in Figure 3.3, determines how many upstream dike segments are hydraulically affected and directly influences the scope and cost of reinforcement projects.

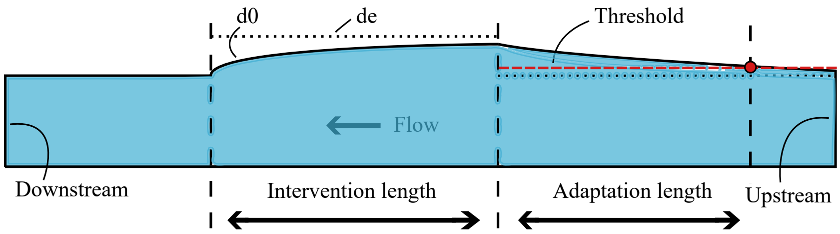

Figure 3.3: Backwater curve effects due to outward dike reinforcement visualised

Backwater effects are estimated using an empirical approximation of Bresse's analytical solution (Equation 3.2). This formulation assumes subcritical flow in a rectangular, uniform channel [Blom, 2025a], which does not fully reflect the compound geometry of the current model, comprising both a main channel and a floodplain. In this equation, d represents the actual water depth at a given location, d_e the equilibrium depth, and d_0 the initial water depth. The term $x - x_0$ denotes the horizontal distance from the start of the intervention (e.g. the outward dike reinforcement), and $L_{1/2}$ is the half-backwater length, determined using Equation 3.3.

In this study, the equilibrium water level used in the empirical approximation is that of the floodplain, which closely resembles the actual compound channel equilibrium depth, though it is slightly lower. While the maximum WLD remains the same as if the channel's equilibrium depth were used, the larger $d_0 : d_e$ (see Equation 3.3) ratio results in longer half-backwater lengths and thus slightly more conservative (i.e. greater) WLD at the end of the intervention.

$$d(x) = d_e + (d_0 - d_e)\ e^{\left(\frac{x-x_0}{L_{1/2}}\right)} \quad (3.2)$$

$$L_{1/2} = 0.24\ \alpha\ \frac{d_e}{i_b} \left(\frac{d_0}{d_e}\right)^{4/3} \quad (3.3)$$

Despite this discrepancy, the empirical fit is retained, as an alternative exponential formulation yielded unrealistic adaptation lengths and was therefore rejected.

To address the mismatch between the 1D model and the assumptions underlying Bresse's formulation, a calibration factor α is introduced for both the intervention and adaptation lengths in the half-backwater length equation (Equation 3.3). This factor is derived from D-Hydro Suite simulations, which more accurately capture the complex hydraulic behaviour of the Waal River. The calibration ensures that the simplified model remains consistent with the numerical reference and compensates for the limitations of Bresse's analytical approach.

The resulting calibration factors are presented in Table 3.1. Their substantial values reflect a clear discrepancy between the 1D model and the D-Hydro simulations, justifying the need to fit the simplified backwater formulation to better match the Waal's hydraulic behaviour. This discrepancy partly results from structural model differences, but also from the flexibility allowed during calibration, particularly at higher discharges. Nevertheless, the factors are retained to ensure conservative estimates, in line with the approach outlined in Chapter 1. Further details are provided in Appendix G.

Table 3.1: Calibration factors α used in the half-backwater length equation, with distinct values applied for the intervention length and the adaptation length.

	Calibration factor α
Intervention length	1.49
Adaptation length	1.90

Definition of the adaptation end
According to the simplification of Bresse's solution of the Bélanger equation, which is valid for backwater effects that are much smaller than the water depth, backwater effects become negligible beyond four times the half-backwater length, at which point only a small fraction of the WLD remains [Blom, 2025a]. However, as shown in Appendix D, this cut-off based on a relative backwater effect diverges significantly from the absolute threshold for the effect implied by the RBK, i.e., the 1-mm rule. Applying an even stricter threshold further extends the adaptation length, with implications on how far upstream dikes are affected, as discussed in Section 4.2.3.

In line with the RBK, this thesis adopts an absolute threshold-based definition to identify the end of the backwater effect. The backwater effect is considered negligible when the water level deviates less than 1 mm from the original equilibrium depth (Figure 3.3). This approach also facilitates calibration with the D-Hydro Suite simulations, whose spatial extent is insufficient to determine adaptation lengths directly, as elaborated in Appendix G.

Model input parameters for the Waal River
Building on the context introduced in Chapter 1, Table 3.2 presents the hydraulic variables and dimensions used in the 1D model. The list below provides further clarification on the assumptions and calibration choices underlying these values.

- A maximum discharge of 12,000 m^3/s is used as a stress test for outward expansion. This reflects two-thirds of the Delta Programme's conservative design discharge of 18,000 m^3/s at Lobith, based on flow distribution at the Pannerdensche Kop bifurcation [Warmink et al., 2013, De Vriend et al., 2017].
- Although 18,000 m^3/s is used in policy, the physically realistic upper limit is estimated at 17,500 m^3/s, beyond which flooding in Germany is expected [De Vriend et al., 2017].
- The 1D model was calibrated using a discharge of 17,000 m^3/s; simulations beyond this upper limit may therefore lack full validation.
- Floodplain roughness is expressed as a Chézy value, converted from Nikuradse roughness height using the White-Colebrook formula [Domhof et al., 2018], based on typical vegetation types in the Waal floodplains [Rijkswaterstaat Waterdienst and Grontmij Nederland B.V., 2010].
- Channel roughness is calibrated for Chézy values typically observed during high flows in the Waal [Warmink et al., 2007].
- The channel depth (up to floodplain level) and width are based on realistic ranges but remain fixed throughout the analysis.
- The floodplain width ranges from 500 to 3,500 m, based on observed extremes of the Waal River using Google Earth. [Google Earth, 2024]

- The outward dike reinforcement range (5–20 m) reflects the minimum and maximum expansion (as discussed in Section 2.2). Other values are not considered.
- The maximum intervention length is set to 70 km, corresponding to the length of the Waal from Boven-Merwede to Nijmegen. However, in this study, the 34 km section between Dreumel and Nijmegen is considered as the entire Waal length, excluding tidal dynamics and bifurcation effects.
- A 15 km section is deemed a typical reach for a single reinforcement project [Hoogwaterbeschermingsprogramma, 2025].

When analysing the influence of a single variable, all others are held constant, except for the magnitude of outward expansion.

Table 3.2: Variables used in the simplified 1D compound channel model. Analyses are conducted within the specified range of each parameter. When the influence of a single variable is assessed, all others are held constant unless stated otherwise. Fixed values are based on literature, satellite images or calibration with D-Hydro.

Variable	Symbol	Unit	Range	Fixed value
Discharge	Q	m^3/s	0 - 12,000	12,000
River bed slope	i_b	m/m	–	1.03×10^{-4}
Channel depth	d_c	m	5 - 10	8.2
Channel width	B_c	m	240 - 300	265
Channel Chézy	C_c	$\sqrt{m}/s$	35 - 55	45
Floodplain width	B_f	m	500 - 3500	2100
Floodplain Chézy	C_f	$\sqrt{m}/s$	4 - 41	22
Outward dike expansion	–	m	5 - 20	20
Intervention length	–	m	0 - 70,000	15,000

Using the equilibrium depth from the calibrated 1D model and the water levels derived from the empirical fit to Bresse, the upstream water depth and corresponding adaptation lengths can be determined for various intervention scenarios. This provides a basis for analysing the hydraulic response to different reinforcement strategies.

3.1.2. Model framework: D-Hydro suite model

The D-Hydro Suite is applied in this study as follows:

- Version 2023.02 of the D-Hydro Suite was used, as version 2025.01 was not fully operational on the available hardware.
- The schematisation dflowfm2d-rijn-beno19_6_20m_waal-v2d, provided by Informatiepunt Leefomgeving, was used.
- Baseline 7 was applied in combination with the baseline schematisation baseline-rijn-beno19_6-v2, developed by Rijkswaterstaat and also provided via Informatiepunt Leefomgeving.
- These schematisations were made available by Rijkswaterstaat for research purposes. According to personal communication with Rijkswaterstaat (July 2025), more recent schematisations were not required for this exploratory study.
- Multiple variants of outward dike reinforcement were implemented in Baseline 7, in line with the Basisrapport WBI 2017 [De Waal, 2018, Spruyt et al., 2024]
- In the D-Hydro analysis, the Waal is defined as the stretch between Dreumel and Nijmegen. This region was selected as a feasible domain for applying outward dike expansion, while avoiding the influence of tidal dynamics and preventing possible interference with the Pannerdensche Kop within the model.

Further details on the schematisation and implementation of outward dike expansion in the model are provided in Appendix F. Appendix G elaborates on the data cleaning procedures applied to the D-Hydro outputs before further analysis.

3.2. Results

3.2.1. Floodplain characteristics as drivers of hydraulic sensitivity to ODR

The hydraulic sensitivity of outward dike reinforcement (ODR) is primarily governed by floodplain and channel roughness, floodplain width, and expansion magnitude, as derived from Equation 3.1. These variables influence the equilibrium depth difference resulting from ODR. To assess their individual and combined effects, a sensitivity analysis is performed using the simplified 1D compound channel model.

Roughness

First, the effects of varying channel and floodplain roughnesses are investigated. Figure 3.4 shows how variations in the roughness for the channel and floodplain influence the equilibrium water depth during peak discharge in the Waal River (12,000 m^3/s). When one Chézy value was varied, the other was held constant.

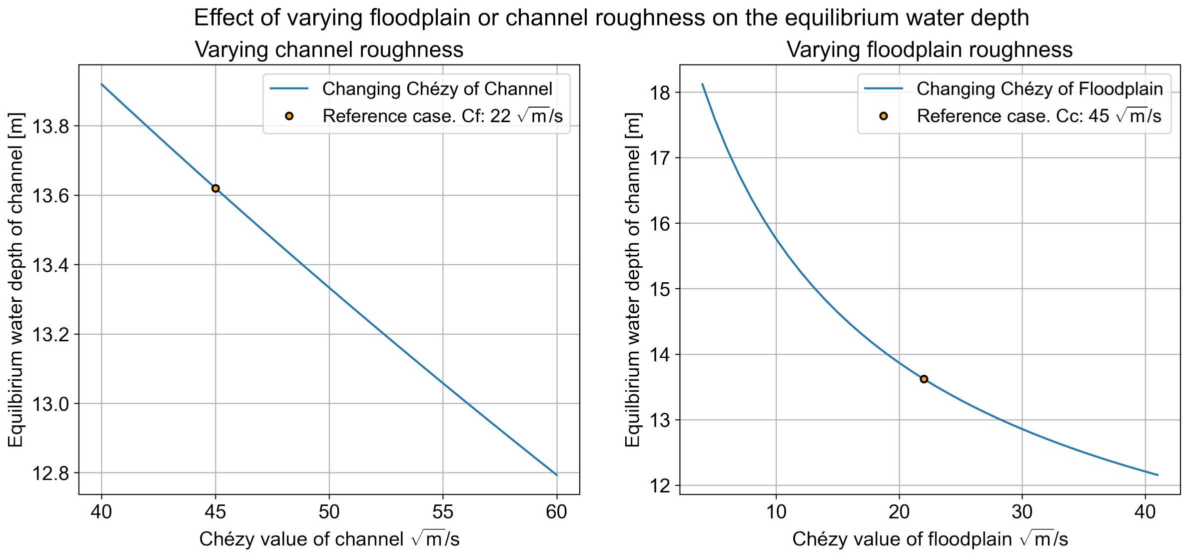

Figure 3.4: Effect on the water level due to varying roughnesses, for the most extreme feasible discharge through the Waal. The reference scenario is based on the calibrated Chézy values.

The plots demonstrate that increasing the floodplain Chézy value (i.e., reducing roughness) results in a more pronounced decrease in water depth than a comparable change in channel roughness. Similarly, higher floodplain roughness leads to greater increases in water levels than equivalent changes in channel roughness. These findings are consistent with the observation that “high roughness values for the main channel do not necessarily result in extreme water levels but are partly compensated by a higher discharge through the floodplain regions” [Warmink et al., 2013, p. 309], as floodplains contribute significantly to conveyance during high-water conditions.

The floodplain Chézy value varies widely from 4 to 41 $\sqrt{m}/s$, depending on vegetation type [Rijkswaterstaat Waterdienst and Grontmij Nederland B.V., 2010], potentially leading to local water level differences of up to 6 metres. Although “uncertainty in spatially distributed vegetation has a minor overall impact on water level uncertainty than main channel roughness” [Warmink et al., 2013, p. 314], extreme cases, such as dense vegetation in high-conveyance areas, can still cause significant deviations observed [Warmink et al., 2013]. In contrast, channel roughness is more constrained (40 - 60 $\sqrt{m}/s$ [Warmink et al., 2007]) and typically results in water level variations of around 1 metre under high-flow conditions.

Given the broader range and more substantial influence of floodplain roughness on the equilibrium depth, its effects are analysed in greater detail in the context of outward dike expansion. Moreover, as discussed in Appendix G, channel roughness is often calibrated in relation to floodplain roughness, further underscoring the importance of the channel roughness.

Floodplain width

Figure 3.5 illustrates how typical floodplain widths observed in the Waal River influence the equilibrium water depth under realistic discharge scenarios.

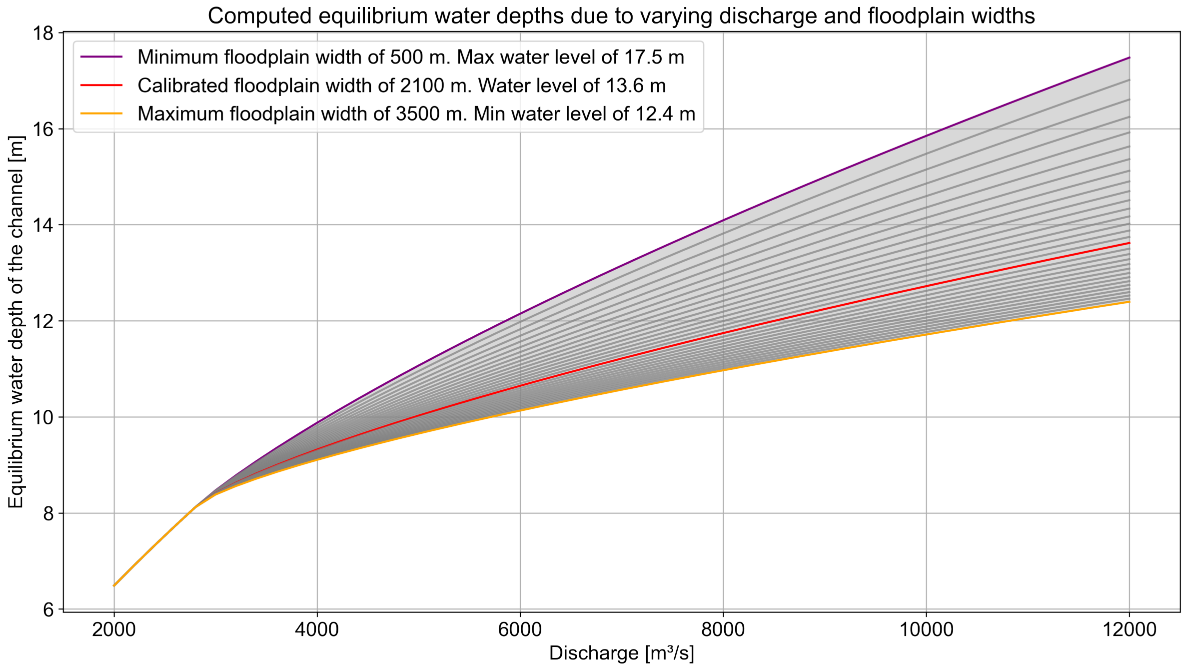

Figure 3.5: Effect of varying floodplain width on equilibrium depth

The figure shows that floodplain begins to affect equilibrium depth only once a discharge threshold of approximately 3,000 m^3/s is exceeded, corresponding to the point at which Zfloodplain is surpassed (see Figure 3.1 in Subsection 3.1.1). Narrower floodplains result in significantly higher water levels, with increases exceeding 16 metres observed.

Floodplain width is therefore a critical factor in assessing the feasibility of outward dike expansion. Smaller floodplains, which are more sensitive to reductions in conveyance capacity, may exhibit stronger hydraulic responses. This is further analysed in Figure 3.6, where a maximum outward expansion of 20 metres is applied to various floodplain widths, and the resulting equilibrium depth differences are plotted.

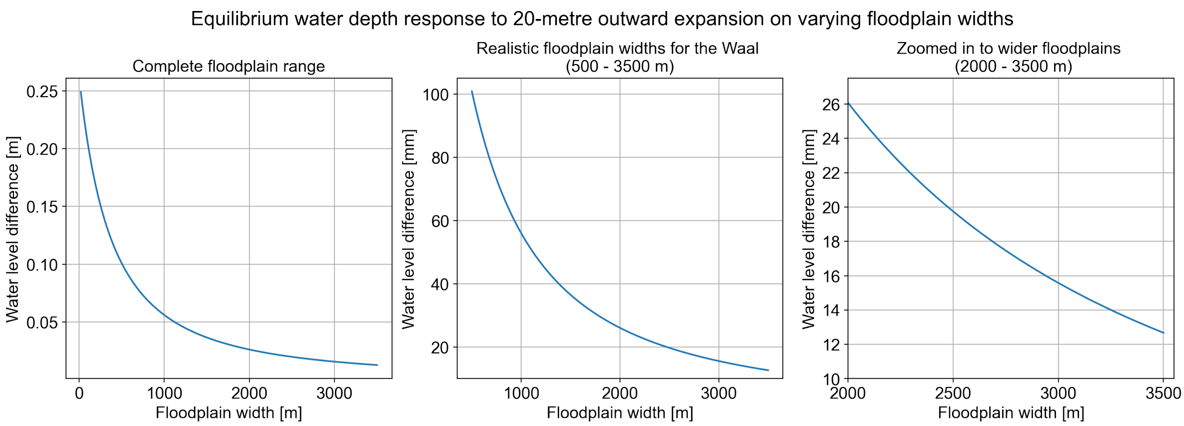

Figure 3.6: Effect of 20-metre outward dike expansion for varying floodplain widths on equilibrium water depth difference.

The results indicate that in narrower floodplains, where the ratio of expansion width to floodplain width is higher, the impact of outward expansion on water level differences is more pronounced. The relationship between equilibrium WLD and the proportion of floodplain claimed follows an approximately logarithmic trend. This highlights the importance of exercising caution when considering expansion in areas with narrow floodplains.

Combined effects on water level differences

While floodplain width and the Chézy coefficient individually influence equilibrium water depths, their combined effect also warrants analysis. Figure 3.7 shows the impact of the maximum 20-metre outward dike expansion across various floodplain widths and roughness values.

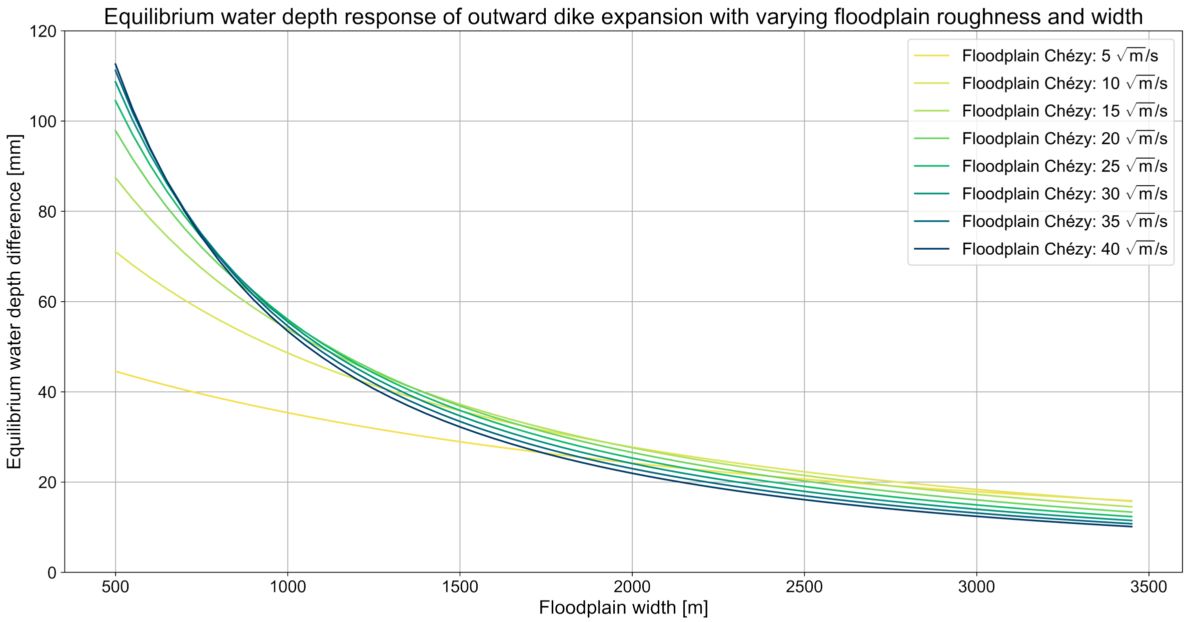

Figure 3.7: Equilibrium water depth differences resulting from a 20-metre outward dike expansion, across varying floodplain roughness and width combinations observed in the Waal River. All other parameters remain fixed as shown in Table 3.2

.

Narrow and smooth floodplain configurations show the highest hydraulic sensitivity to the implementation of ODR, as can be concluded from Figure 3.7. The figure illustrates equilibrium depth differences between scenarios with and without outward expansion, ranging from 1 to 12 cm depending on the floodplain and roughness configuration. Up to a floodplain width of approximately 2,000 metres, the influence of roughness remains limited and smoother floodplains are beneficial, with a modest hydraulic impact that remains below 3 centimetres. However, in narrower floodplains, roughness becomes a dominant factor. Higher Chézy values (i.e., lower roughness) result in greater water level increases, as smoother floodplains convey more flow. When part of a floodplain is reclaimed through outward expansion, the relative loss in conveyance is more substantial, leading to a sharper rise in water levels. Conversely, under rough conditions, equilibrium depths are already significantly elevated, but therefore, the loss of conveyance capacity is less drastic. This is illustrated in Figure 3.8.

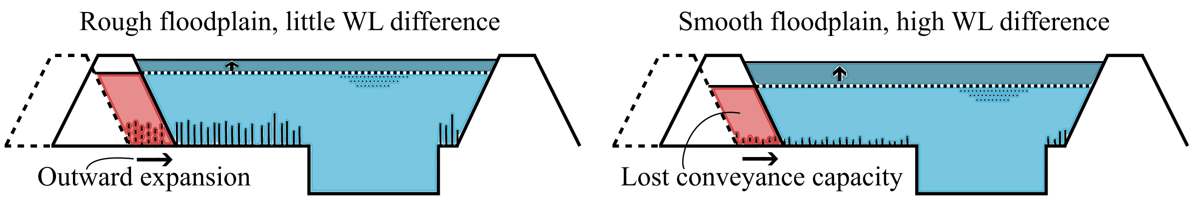

Figure 3.8: Effect of floodplain roughness on water level difference

3.2.2. Location-based hydraulic impact

To assess how the spatial application of ODR influences hydraulic behaviour, this subsection examines location-specific effects using the D-Hydro suite. Three scenarios are considered: application within bottlenecks, outside bottlenecks, and across the full Waal reach (Figure 3.9; for the model schematisations, see Appendix F). The analysis consists of two parts. First, the local hydraulic effects observed in the D-Hydro simulations are examined, including spatial irregularities in water level response. Second, the influence of the location where ODR is implemented is assessed.

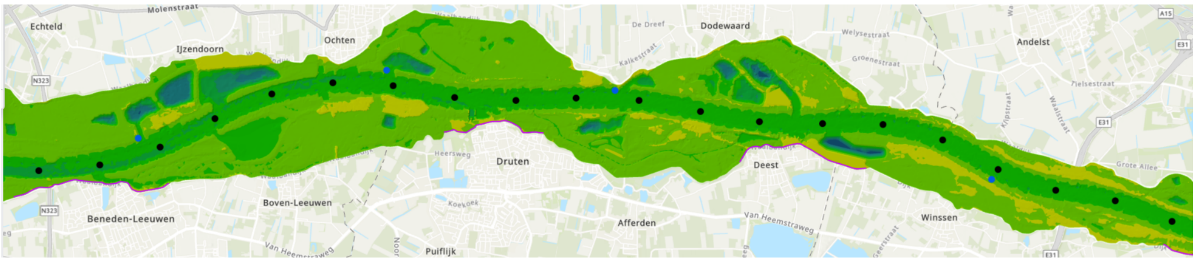

Figure 3.9: The Waal River between river km WL_912 (left black dot) and WL_892 (right black dot). Purple lines indicate ODR application within bottlenecks on the southern Waal bank (simulation variant a11). Variant a10 applies ODR outside these bottlenecks, as the inverse of variant a11. Full-reach application combines both variants from WL_921 to WL_887.

Local effects of ODR observed with D-Hydro

Figure 3.10 shows the WLD resulting from a maximum 20-metre outward expansion. The full-reach scenario (Bordeaux red line) is compared to local effects within (orange) and outside (red) bottlenecks.

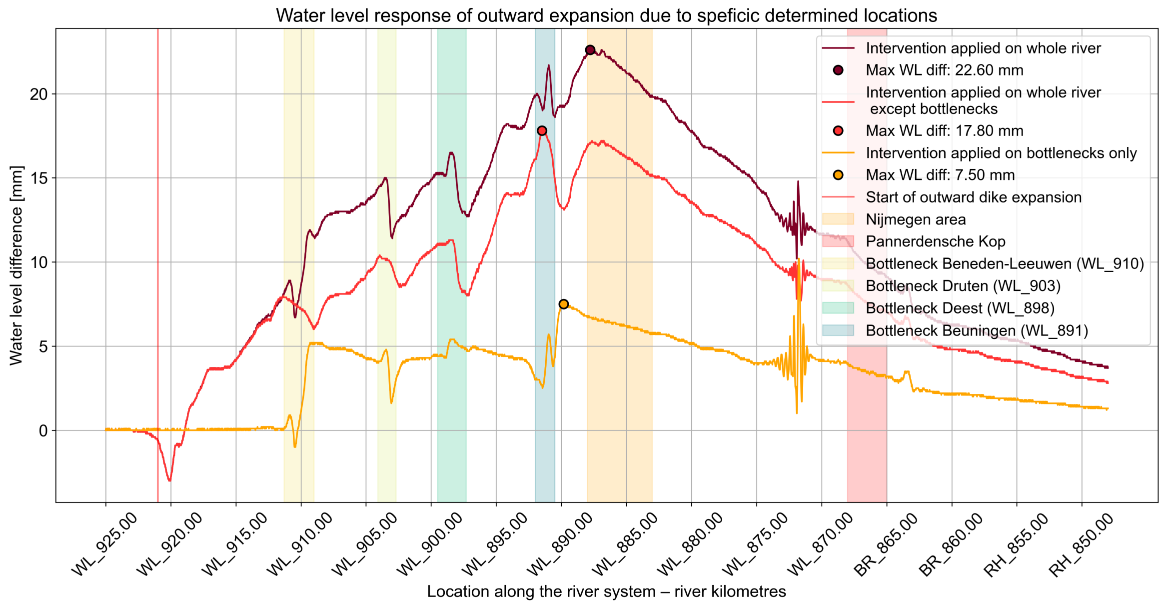

Figure 3.10: Water level difference resulting from ODR implementation at different locations. Coloured segments highlight bottlenecks areas, Nijmegen and the Pannerdensche Kop.

The WLD does not follow a smooth gradient but instead displays a staggered pattern, which is primarily driven by the geometry of the flow regime and the distribution of floodplains, as visualised in Figure 3.11. Between WL_919 and WL_913, near Tiel, the absence of significant floodplains leads to a steep rise in WLD, briefly stagnated by a small increase in floodplain area and the opening of the Amsterdam-Rijnkanaal. Further upstream, widening floodplains after the Beneden-Leeuwen bottleneck reduce the upstream impact, while subsequent narrowing at Ochten (WL_904), Deest (WL_898) and Beuningen (WL_892) causes sharp increases followed by immediate dips. The spatial variability highlights the significance of local floodplain geometry in influencing the hydraulic response and, consequently, the impact of outward dike expansions.

Although local variations in hydraulic roughness are present within the schematisation, these cannot be explicitly attributed to specific hydraulic impacts and are therefore not analysed in detail. The observed variability underscores the importance of floodplain configuration in determining the local hydraulic impact, independent of channel geometry, which remains approximately constant due to navigational constraints.

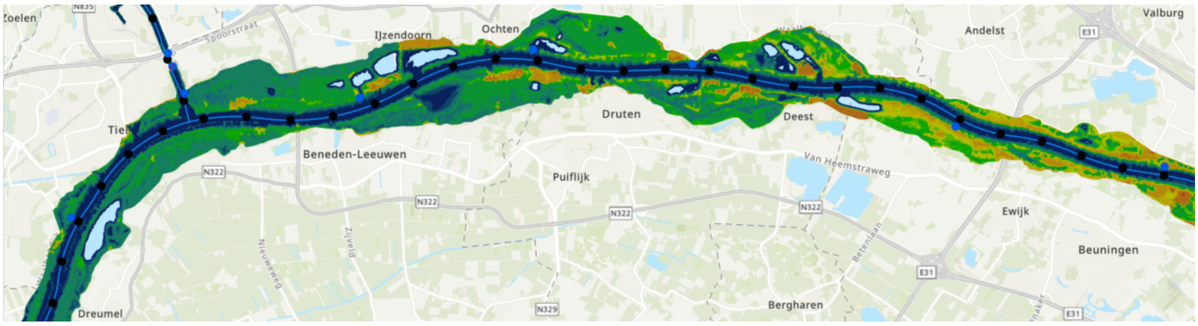

Figure 3.11: Schematisation of the Waal river between Dreumel (WL_921) and Beuningen (WL_890). Visualisation of the Baseline beno19 schematisation via ArcGIS

Just upstream of the intervention start point at WL_921, a local dip in water level is observed over approximately 2 kilometres. This reflects typical backwater effects, which are inherent to such interventions and cannot be fully avoided [Van Doorn and Rijkswaterstaat Water, Verkeer en Leefomgeving, 2023].

At WL_872, a significant wiggle is observed across all runs and persists in subsequent simulations. This irregularity is likely caused by a numerically unstable grid node and is unrelated to the hydraulic effects of ODR. However, it influences the adaptation lengths determined with the D-Hydro simulations. Further details are provided in Appendix G.

Location dependency of implementing ODR

The hydraulic impact of ODR strongly depends on its spatial application, with continuous reinforcement producing the highest water level differences. The maximum observed increase is 22.6 millimetres, corresponding to a 20-metre expansion applied along the entire southern Waal bank (Figure 3.10). Simulations with discrete reinforcement, either within or outside bottlenecks, lead to lower overall water level differences.

When the water level differences from staggered interventions at bottleneck locations are summed, the resulting peak exceeds that of the continuous scenario. This behaviour reflects the dynamics of backwater curves: in staggered interventions, the flow locally returns to its original equilibrium depth before adjusting again, leading to steep rises at transition points. These local drops followed by sharp increases illustrate the system's response to alternating equilibrium conditions.
However, despite these local peaks, none of the staggered interventions individually exceed the total WLD observed in the fully reinforced scenario. This confirms that while short interventions are penalised, continuous reinforcement over longer stretches consistently produces the highest WLD.

The simulations in Figure 3.10 indicate that ODR applied within bottlenecks has the most pronounced local hydraulic effect. The figure shows steeper water level rises at bottleneck locations such as Beneden-Leeuwen and Beuningen. At Deest, a sharp drop is observed in the unreinforced scenario, which does not occur when expansion is applied at the bottleneck. This might indicate that the equilibrium depth has been reached for the scenario with reinforcement in the bottleneck. The significant drop observed near Druten cannot be fully explained, but it is hypothesised to result from the channel widening following the bottleneck on the northern bank near Ochten, where the WLD of variant A10 (red line) also shows a marked decrease.

3.2.3. Impact of outward expansion magnitude

The hydraulic response to different expansion magnitudes across the considered Waal stretch with the D-Hydro suite simulations is shown in Figure 3.12. The maximum observed water level difference is 22.6 millimetres, corresponding to the maximum outward expansion of 20 metres (Bordeaux line). As expected, smaller expansions result in proportionally lower water level differences. These findings confirm that the overall water level response remains within the order of several centimetres, but do not reach the significant equilibrium water level difference observed in Figure 3.7 in Subsection 3.2.1.
From the figure, it can be concluded that the dip observed just upstream of the intervention start point (between WL_921 and WL_919) is governed by the amount of outward expansion.

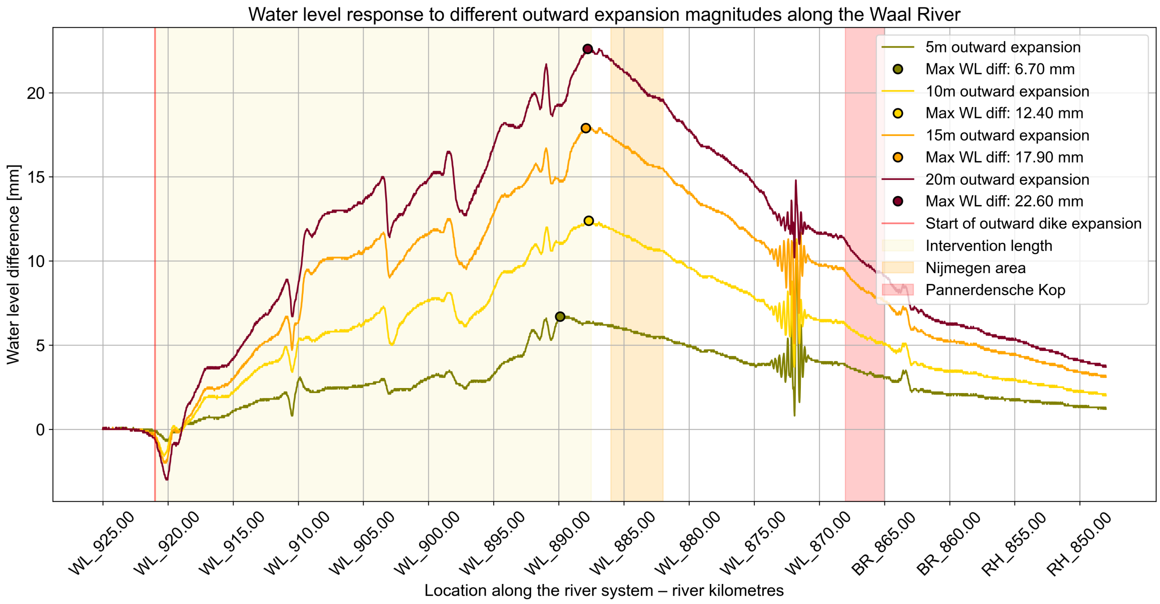

Figure 3.12: Water level response due to varying outward dike expansion magnitudes in the Waal flowing regime between Dreumel and Nijmegen, computed with the D-Hydro suite model. The expansion magnitudes range from 5 metres (olive) to 20 metres (Bordeaux), with steps of 5 metres.

To identify the overarching trend in the water level response to expansion over the whole southern Waal bank, the D-Hydro suite is complemented by the simplified 1D model. Figure 3.13 shows the increase in WLD for the combination of averaged floodplain widths and expansion magnitude.

The relationship between outward expansion magnitude and water level difference appears approximately linear, as shown in Figure 3.13. For each line corresponding to a specific floodplain width, the increase in WLD between 5 and 20 metres of expansion follows a linear trend, although the slope varies. Therefore, interpolation within this range for a reach with a single averaged floodplain width provides a reasonable estimate of the expected hydraulic impact.
This linear relation also appears to be reflected in the D-Hydro suite simulations in Figure 3.12, where the maximum water level differences increase in roughly 5 mm increments: 6.7 mm for 5 m expansion, 12.4 mm for 10 m, 17.9 mm for 15 m, and 22.6 mm for 20 m. While this suggests an approximately proportional trend, the spatial response shows localised deviations, indicating that interpolation should be applied with caution, particularly in regions with strong local hydraulic effects.

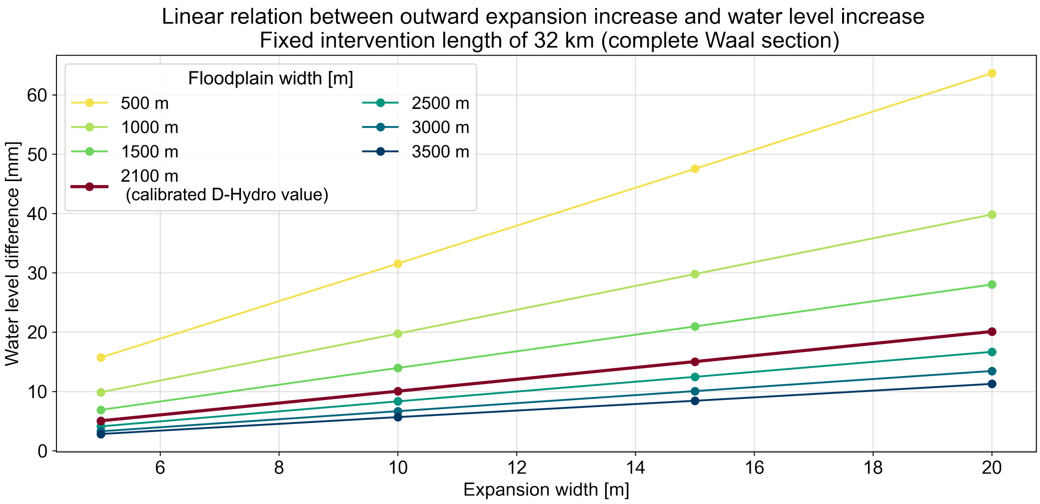

Figure 3.13: Relationship between outward expansion width and water level difference for various floodplain widths, computed with the 1D model. As discussed in Subsection 3.2.2, smaller floodplains result in higher water level differences. The intervention length is fixed at 34 km, covering the entire southern Waal bank. The 2100 m floodplain represents the calibrated average from D-Hydro simulations; other parameters are fixed as listed in Table 3.2.

3.2.4. Intervention length dependency

As previously theorised with the simplified 1D model framework (Subsection 3.1.1), water level differences are influenced by the intervention length of outward dike reinforcement. This effect is further explored using D-Hydro simulations for the maximum 20-metre expansion, as shown in Figure 3.14.

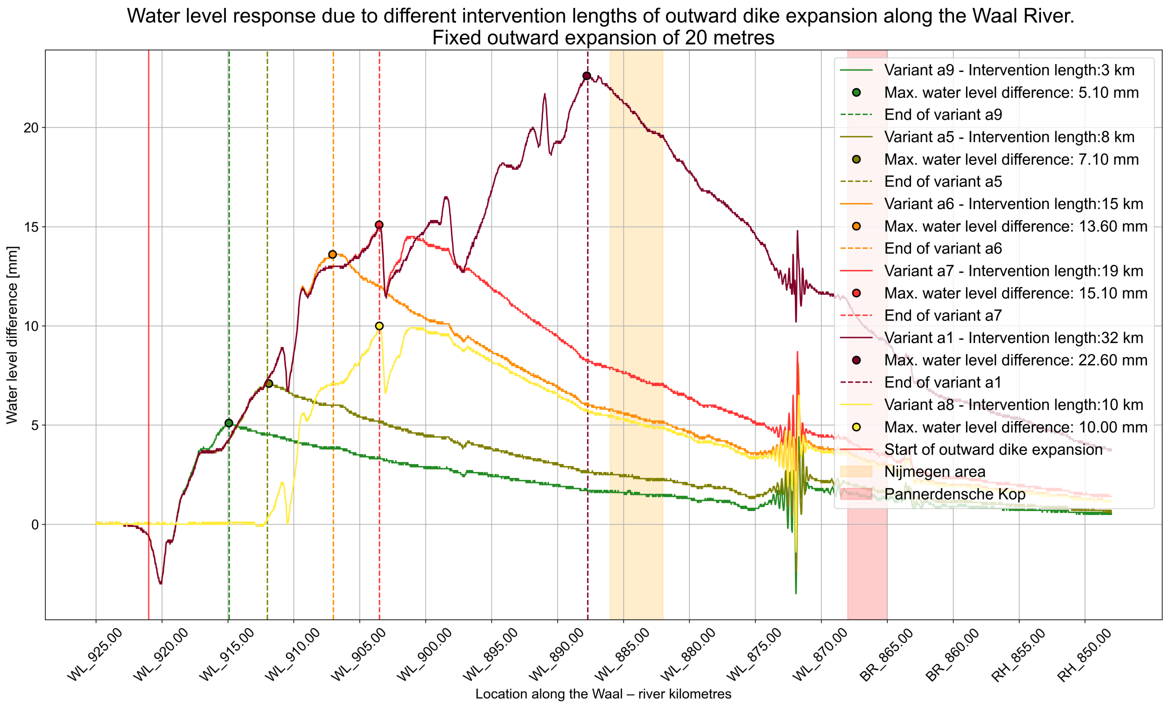

Figure 3.14: Maximum water level differences resulting from 20-metre outward dike expansion, shown for varying intervention lengths. Results computed with the D-Hydro suite model.

Figure 3.14 demonstrates that longer intervention lengths result in greater water level differences, with the maximum difference consistently observed at the upstream end of the outward intervention. This aligns with expectations: a longer stretch provides more distance for the flow to approach the new equilibrium depth, until the intervention is completed. However, even for the longest simulated intervention (Bordeaux line), full flattening of the WLD is not observed, indicating that reaching the equilibrium water depth requires considerably greater lengths than those currently modelled.

Moreover, short interventions are disproportionately penalised, as the steepest water level rise occurs within the first few kilometres of reinforcement. This initial response is non-linear, meaning that shorter stretches experience relatively high water level differences per unit of intervention length, without benefiting from the gradual stabilisation seen in longer interventions. The gradual flattening of the water level response is also influenced by floodplain geometry, as discussed in Section 3.2.2.

The inevitable decrease in water level observed at the start of the intervention (at WL_921, seen from downstream) shows no variation across different intervention lengths. However, the dip does vary with the magnitude of outward expansion (Figure 3.12), suggesting it is governed by the local water level near the transition zone rather than by the maximum water level reached upstream.

3.2.5. Hydraulic impact of ODR: Design graphs and implications

This subsection presents graphs to estimate the hydraulic impact of ODR across a range of design variables, using 5-metre and 20-metre outward expansions as bounding cases. These graphs serve as engineering tools to support design decisions to apply ODR in contexts without strict regulatory constraints, as introduced in the thesis objective in Section 1.4.

The 1D model is applied to analyse combinations of averaged floodplain geometry and intervention length, to approximate the WLD at the upstream end of the intervention. Previous subsections have shown that these variables are the primary drivers of the hydraulic impact and are therefore examined within realistic bounds for the Waal River. The resulting graphs provide a first-order estimation of the hydraulic impact. For expansion widths between 5 and 20 metres, interpolation between the graphs serves as a practical method to estimate the water level differences, given the approximately linear relationship between expansion and hydraulic response.

Additionally, a graph estimating the adaptation length resulting from 20-metre ODR is derived using the 1D model, based on identical input parameters and design ranges. This provides a first approximation of the expected adaptation length, determined according to the absolute threshold defined in Subsection 3.1.1.

Hydraulic response to 20-metre outward expansion

Figure 3.15 presents the estimated water level difference resulting from the maximum outward expansion of 20 metres, for varying intervention lengths and averaged floodplain widths. If both riverbanks are reinforced simultaneously, the effect should be considered cumulatively. When applied sequentially, the combined impact can be approximated as a single continuous intervention.

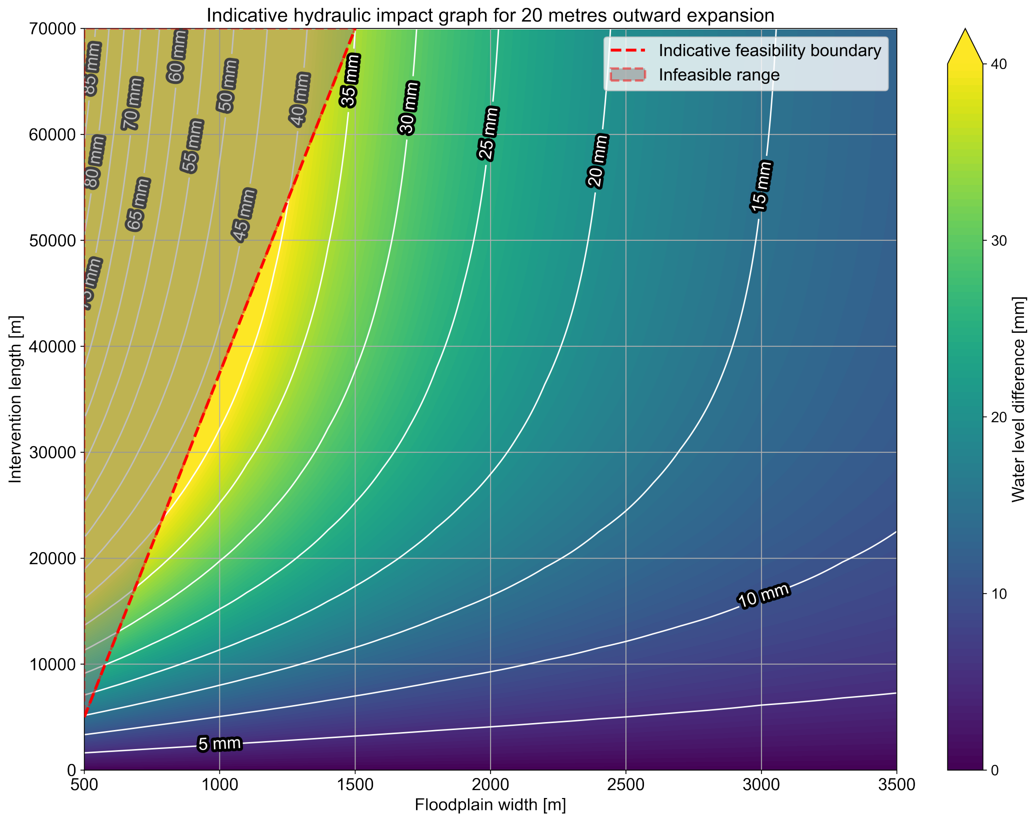

Figure 3.15: Water level difference due to varying intervention lengths and average floodplain widths, for the maximum 20-metre outward expansion. The graph is calibrated for the Waal River, assuming an average roughness of 22 $\sqrt{\mathrm{m}}/\mathrm{s}$. The x-axis represents the averaged floodplain width, covering a realistic range observed in the Waal. The y-axis indicates the intervention length along the river reach, with 70 km being the maximum considered reach length. The indicative feasibility boundary marks the combinations of local intervention lengths and averaged floodplain widths that are considered realistic based on Google Earth observations.

Under the upper-bound scenario of a 20-metre outward expansion, the maximum expected WLD within a realistic design domain ranges from 0 to 4 centimetres. This range reflects combinations of floodplain width and intervention length that are considered realistic based on satellite imagery of the Waal River.
Although the full computational domain spans approximately 0 to 9 cm, not all combinations of the input bounds are realistic. In particular, extreme configurations, such as long intervention lengths combined with very narrow floodplains, are unlikely to occur in practice. While narrow floodplains may exist locally (e.g. at bottlenecks), over longer stretches the average floodplain width typically exceeds 1,000 to 2,000 metres [Projectteam Meanderende Maas, 2024, Google Earth, 2024]. The realistic feasibility boundary has been manually defined based on satellite imagery, and is indicated in the accompanying graph.

Figure 3.15 further reveals a non-linear relationship between intervention length and water level difference: the hydraulic impact increases rapidly over the initial kilometres, but gradually approaches an equilibrium state. This pattern aligns with the findings presented in Section 3.2.4, and highlights the penalisation of short ODR interventions.

Hydraulic response to 5-metre outward expansion

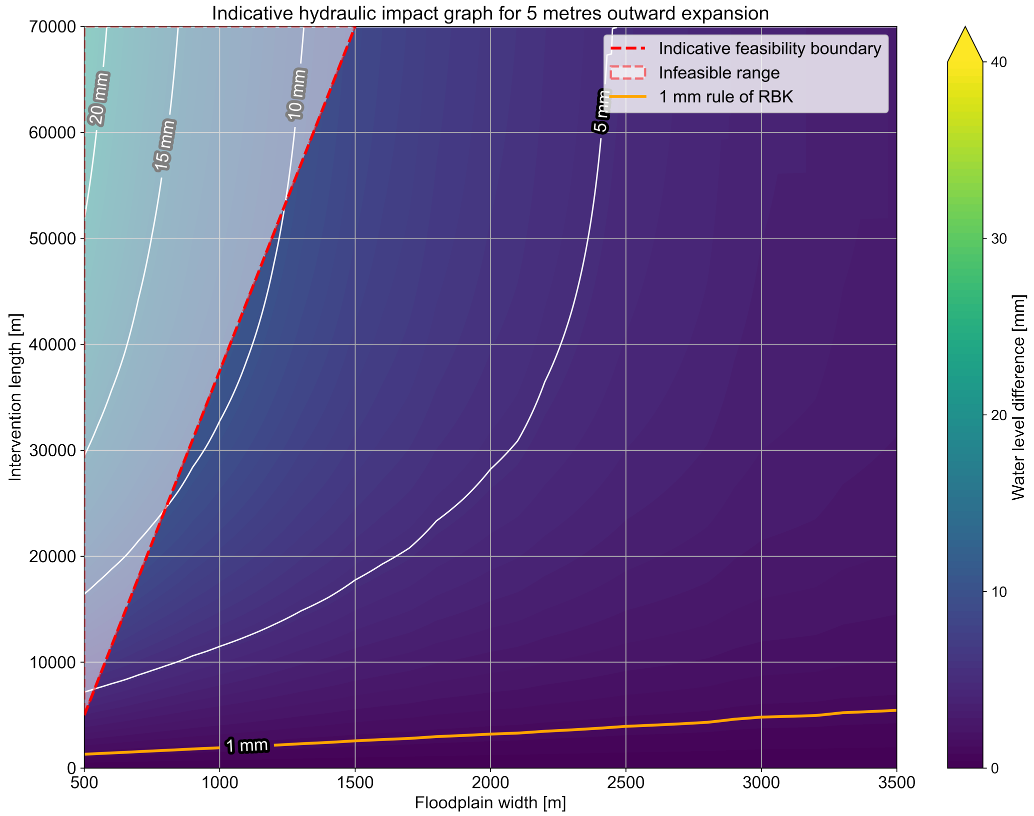

Figure 3.16: Water level difference for the same ranges of intervention length and floodplain width as in Figure 3.15, but based on the minimum 5-metre outward expansion. The graph is calibrated for the Waal River using the same modelling assumptions.

The lower-bound hydraulic response resulting from the minimum 5-metre outward expansion remains below 2 centimetres across most realistic design scenarios along the Waal, as illustrated in Figure 3.16. In certain combinations, the 1-millimetre threshold defined by the RBK is not even exceeded. Similar to the 20-metre case, a steep initial increase in water level is observed for short intervention lengths, highlighting the relative inefficiency of short interventions.

Adaptation lengths of 20-metre outward expansion

Figure 3.17 shows the expected adaptation lengths for combinations of averaged floodplain widths and intervention lengths, based on the maximum outward expansion of 20 metres. These results are linked to the water level differences shown in Figure 3.15. Note that the adaptation length approximations are based on a conservative modelling and calibration (Section 3.1.1 and Appendix G). Furthermore, a threshold of 0.1 mm is applied in this graph, reflecting an exceptionally strict and conservative enforcement criterion. Practitioners should interpret the graph with an understanding of these underlying assumptions.

Even minor water level increases, in the order of a few centimetres, can result in adaptation lengths extending over several tens of kilometres, as illustrated in Figure 3.17 For very narrow averaged floodplain widths, the adaptation length increases steeply with growing intervention length. One must consider that the amount of WLD resulting from the implementation of ODR will affect the magnitude of the adaptation length. Both the magnitude and length of outward expansion influence upstream effects, with greater magnitudes and longer intervention lengths resulting in significantly longer adaptation lengths

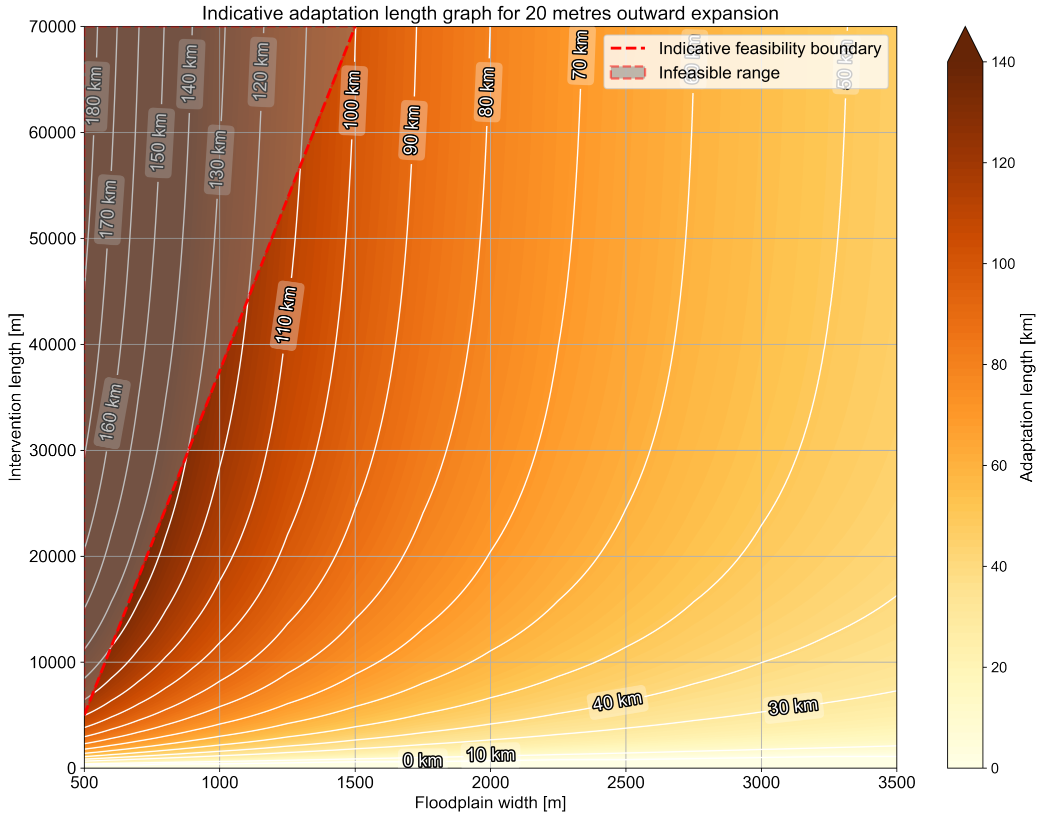

Figure 3.17: Maximum adaptation lengths due to 20-metre outward dike expansion for the same ranges of intervention lengths and floodplain widths as in Figure 3.15. The graph is calibrated for the Waal River using the same modelling assumptions. The adaptation length endpoint is defined by the absolute threshold of 0.1 mm difference compared to the original equilibrium water depth.

The results are sensitive to how the endpoint of the adaptation length is defined. The graph presents adaptation length estimates stricter than the 1-mm rule by the RBK. This level of precision is justified in contexts with tight water level tolerances, such as in the Waal River, where almost no change in discharge distribution is permitted upstream at Pannerdensche Kop [Van Doorn and Rijkswaterstaat Water, Verkeer en Leefomgeving, 2023]. However, it could be argued that this threshold should be more flexible, depending on the context. Since the observed adaptation lengths are highly sensitive to small water level differences (WLD), such choices have a substantial impact on the resulting estimates. If for example the relative threshold based on the simplified empirical fit solution by Bresse were applied instead, adaptation lengths would be shorter and less steep. Nevertheless, these remain in the order of 50 kilometres (Appendix D).

3.3. Main insights on the hydraulic impact by ODR

The expected water level difference (WLD) due to maximum outward dike reinforcement (ODR) in the Waal River ranges from 0 to 4 cm, depending on the application and floodplain characteristics. In the specific Waal reach considered, a 20-metre outward expansion results in a WLD of 22.6 mm, which is used in subsequent chapters. The relationship between expansion and WLD is approximately linear, with applications in hydraulic bottlenecks yielding the greatest impact. For floodplains narrower than 2 km, smooth conditions lead to significantly higher WLD than those with high roughness. Continuous reinforcement causes higher WLD than discrete interventions, although short interventions are penalised by steep local water level gradients, resulting in high WLD per unit of intervention length. Adaptation lengths required to absorb WLD effects span several tens of kilometres, but are highly sensitive to how the endpoint is defined.

4

Feasibility assessment based on affected dike performance

4.1. Approach

This chapter assesses the feasibility of outward dike reinforcement (ODR) by analysing its impact on the performance of existing flood defences. It addresses the second sub-question of the thesis and builds upon the hydraulic effects quantified in Chapter 3. Specifically the 22.6 millimetres water level difference, as determined in Subsection 3.2.3. The feasibility assessment focuses on whether affected dikes remain structurally and functionally adequate under the altered hydraulic conditions, as their performance could impose critical constraints on the applicability of ODR in practice. The feasibility assessment does not concern the structural integrity of the ODR variant itself. The regulations of the RBK are not considered in the analyses.

To assess whether ODR compromises the safety of affected dikes, the primary failure mechanisms are first identified. These are evaluated based on the conceptual dike designs introduced in Chapter 2, which are considered representative cases of dikes exposed to the altered hydraulic impact. The analysis determines whether these mechanisms are significantly affected and whether they may lead to immediate failure or a reduction in functional lifetime.

If the analysis indicates that ODR implementation leads to premature failure of one or more mechanisms, a functional lifetime reduction (FLR) analysis is conducted to quantify the long-term performance impacts. This analysis assumes a predefined design lifetime and estimates the reduction based on expected hydraulic loads and local conditions. Sensitivity analyses are performed to assess the robustness of the results. Based on these outcomes, mitigation strategies are proposed to evaluate whether the original design lifetime can be retained and whether the negative effects of ODR can be effectively addressed in practice.
The chapter concludes with a synthesis of the findings and their implications for the practical feasibility of ODR implementation.

4.1.1. Failure mechanisms selection and formulations

The analysis of structural and functional integrity of affected dikes focuses on three of the four primary dike failure mechanisms: overflow and overtopping, macro-stability (inward), and backward internal erosion [Van Mierlo et al., 2007, Groenewoud, 2016], as illustrated in Figure 4.1. Macro-stability (outward) is excluded from this analysis, as rapid drawdown conditions following a flood are not considered (see Appendix E.2 for further justification). Other failure mechanisms, such as erosion and micro-stability, are also omitted due to their strong dependence on detailed design characteristics, which fall outside the scope of this analysis.

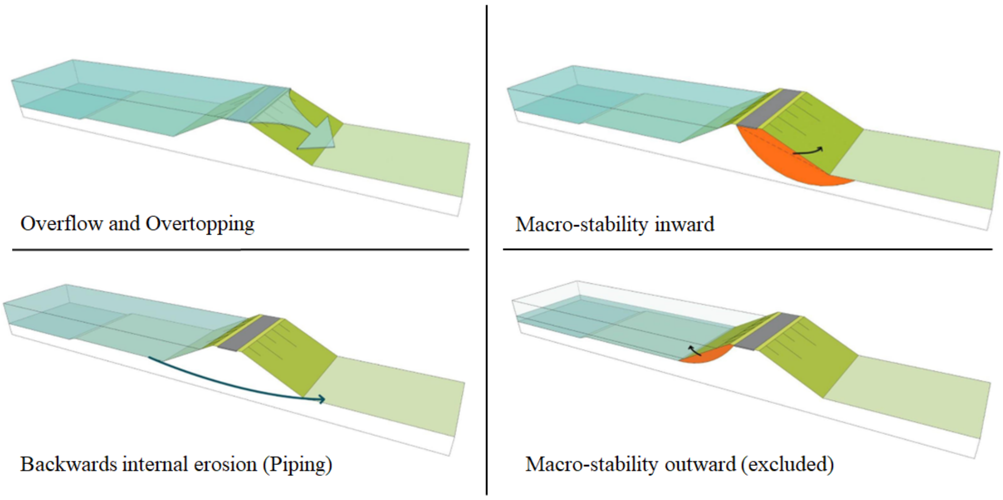

Figure 4.1: The four main failure criteria for dikes [Maronier et al., 2018a]

Overflow and Overtopping
Overflow is defined as the failure mechanism in which the still design water level (DWL) exceeds the crest level, assuming no overtopping freeboard or surplus height is considered. Any occurrence of overflow is considered a failure. Therefore, the crest height must be at least equal to the design water level to satisfy the overflow criterion. Overflow is assessed using a simple limit state function $Z = R - S$, where R is the crest height and S the still design water level (DWL).

For the overtopping failure criterion, a limited volume of water per unit of time per metre is permitted to pass over the dike crest. When this threshold is exceeded, the dike no longer fulfils its retention function, which constitutes the overtopping failure mechanism [Van der Meer et al., 2002]. ODR increases the water depth above the floodplain, potentially promoting more wave growth and thus additional overtopping.

The amount of overtopping depends on wave characteristics and dike design, which together determine the required freeboard height to retain a sufficient overtopping discharge. Wave characteristics are computed using the empirical Bretschneider wave growth formulas. To assess the hydraulic impact of ODR on the required freeboard height, the empirical TAW formula is applied and adapted to evaluate the new crest elevation corresponding to a fixed overtopping discharge. If the required freeboard height must be increased, it is assumed that ODR results in failure.
The parameters used in the analysis reflect realistic yet conservative values for the Waal River and are based on the reference location at Ochten (Chapter 2), the case study (Appendix C), literature and satellite images. The parameters are shown in Table E.2 in Appendix E.1.

The water depths applied in the wave equations are obtained from the simplified 1D hydraulic model, which is specifically used to isolate the effect of water level differences (WLD) on overtopping. The maximum outward expansion of 20 metres is imposed across a range of floodplain widths, and this expansion is subtracted from the fetch length compared to the reference scenario. Far upstream of the outward reinforcement, the fetch is no longer affected by the expansion, but the associated WLD is also reduced. Therefore, only the reduction in fetch length is considered in this analysis. For each floodplain configuration, the model computes the corresponding new equilibrium water depths, which is an conservative approach. These depths do not directly correspond to the hydraulic load level indicated in the conceptual dike designs (Section 2.2). The depth above the floodplain is used instead of depth of the channel, as shallower depths more strongly limit wave growth [Van der Meer, 2002]. For further details on the 1D model setup, refer to Section 3.1.1.

A more detailed explanation overview of the applied formulas for the overflow and overtopping mechanisms are provided in Appendix E.1.

Stability

Dike stability issues arise when the passive resistance moment of the hinterland is insufficient to counteract the driving moment of the dike core. As discussed in Section 2.1, this analysis does not include a detailed stability analysis, as such assessments require advanced geotechnical methods. Therefore, the implications of ODR on stability is assessed using expert judgment and practical engineering heuristics. More elaboration on the stability mechanism is presented in Appendix E.2.

Backward internal erosion (piping)

The internal erosion failure mechanism consists of three sub-criteria: piping, heave, and uplift. These limit states form a parallel system, meaning that failure due to internal erosion only occurs if all three are exceeded simultaneously ['t Hart et al., 2016, Jonkman et al., 2021]. Reinforcing any one of these sub-criteria therefore increases the overall reliability against internal erosion. Uplift is not considered in this thesis, as justified in Appendix E.3

To assess the sub-mechanisms piping and heave, the limit-state formulations proposed by Sellmeijer are applied to the four conceptual dike designs. These designs are used to make the effects of ODR on internal erosion mechanisms tangible, while spatial and soil variations significantly influence the response of the mechanisms. Uniform subsoil conditions and geometric assumptions are applied across all designs, without variation in river flow direction. These assumptions are listed in Table E.3, located in Appendix E.3, where their sensitivity is also discussed.
Seepage lengths are assumed to remain fixed from the outer toe to the inner toe, as described in Section 2.2, without considering specific foreshore or hinterland seepage. This results in a conservative application of the seepage length.
Overall, the assumptions regarding internal erosion are conservative. However, such approaches are commonly applied in practice during preliminary investigations, particularly when soil properties have not yet been fully verified (C. Spoorenberg, personal communication, May 2025).

In the reference scenarios where the WLD is not yet imposed and one of the conceptual designs fails due to either piping or heave, a sheet pile is assumed to be installed to ensure sufficient performance. The required sheet pile length is determined based on the sub-mechanism that results in the shortest length. In this analysis, as is commonly observed in practice, the governing sub-criterion is heave (C. Spoorenberg, personal communication, May 2025) The evaluations explicitly compares different approaches to specifying sheet pile length: one variant applies millimetre-level precision, while another rounds lengths to half-metre increments, reflecting common contractor practice (P. van der Scheer, personal communication, April 2025). The governing limit states for the reference scenario are presented in Table 4.1.
Note that the construction-based alternative in the reference case includes a sheet pile extending 5 metres into the aquifer for enhanced stability (see Section 2.2). For the purpose of internal erosion analysis, this feature is temporarily disregarded, as if it needs to be tested against the relevant limit states.

A more detailed discussion of the applied formulas, the treatment of each sub-mechanism, and the calculation of sheet pile lengths and initial limit states (prior to WLD) is provided in Appendix E.3.

Table 4.1: Overview of seepage lengths and sheet pile requirements for the four conceptual dike designs, based on the sub-mechanism resulting in the lowest required sheet pile length

	Inward reinforcement	Construction-based	Outward 20-metre	Tuimeldijk
Seepage length [m]	55.4	44.4	62.9	52.1
Z-limit state piping	-1.33	-1.81	-1.01	-1.48
Z-limit state heave	0.025	-0.062	0.071	0.002
Exact Sheet pile length [m]	0.00	2.39 (Z heave: 0.001)	0.00	0.00
Rounded Sheet pile length [m]	0.0	2.5 (Z heave: 0.022)	0.0	0.0

4.1.2. Method for approximating functional lifetime reduction

To quantify the reduction in functional lifetime, the exceedance-based limit state formulation is applied to each failure mechanism affected by ODR: $Z = R - S$, where R denotes the resistance of the considered mechanism and S the corresponding load [Kok et al., 2016].
The functional lifetime is defined as the period from dike installation until the year in which the limit state Z becomes negative, indicating that the resistance is no longer sufficient to withstand the load. If ODR accelerates the moment at which this limit state is exceeded, the difference between the intended design lifetime and the actual lifetime is considered the functional lifetime reduction (FLR).
However, if the limit state is only exceeded after the intended design lifetime, i.e., the dike is overdesigned, the full FLR may not be relevant. In such cases, only the portion of the FLR that falls within the intended design lifetime is considered critical. This shortage in functional lifetime reflects the actual impact of ODR on the dike's performance within its design horizon. Therefore the functional lifetime shortage is used as the relevant metric for robust dikes.
The analyses are only conducted in cases where no immediate failure occurs, as identified in the preceding failure mechanism analysis (Section 4.2.1).

For the overflow and overtopping mechanisms, the resistance (R) is defined as the crest height, which decreases over time due to subsidence, settlement, and compaction, as discussed in Section 2.1. For the assessment, the initial crest height is set to meet the required hydraulic load level (HBN) at the end of the original design lifetime, including a height surplus that exactly matches the expected subsidence during this period. For the other mechanisms, the resistance is based on the formulations discussed in Section 4.1.1.

The load S corresponds to the hydraulic load level associated with the ultimate limit state (ULS). For the Waal River, this is defined as a flood probability of 1 in 10,000 per year [Kok et al., 2016]. This HBN increases over time due to climate-change-induced stressors. If outward dike reinforcement introduces a water level difference, this is superimposed on the HBN and more extensively elaborated later in this section.

The hydraulic boundary conditions and geotechnical parameters used to determine the resistance are location-specific. This analysis applies values that are representative of conditions along the Waal River, with specific approximations based on the floodplain near Ochten. These assumptions are discussed later in this section and detailed in Appendix E.

Hydraulic load levels with Hydra-NL

The HBN for each year is computed with the Hydra-NL software, which is in accordance with the 'Wettelijk Beoordelingsinstrumentarium' (WBI2017) [Informatiepunt Leefomgeving (IPLO), 2025]. Hydra-NL is applied as follows:

- Model version v2.8.2 is used.
- Hydraulic boundary conditions are sourced from the IPLO-provided file DPA-Riv_Rijn_oever_2015_ref _S10_DM1p1p12_v02I.sqlite, covering the Waal and adjacent river sections
- Each dike profile calculation in Hydra-NL assumes a 1:3 grass-covered slope, a critical overtopping discharge of 10 l/s/m, and a maximum Rhine discharge of 18,000 m^3/s, as also discussed in Section 2.1.
- All calculations incorporate model and stochastic uncertainties, as required by WBI2017 [Smale, 2018].
- Long-term impacts of water level changes are assessed using Hydra-NL's design mode ('ontwerpmodus'), based on the 'Ontwerpinstrumentarium 2014' (OI2014).

The W+ scenario, which closely aligns with the "Stoom" scenario from the 2024 Delta Climate Scenarios, is adopted in this analysis to evaluate the reduction in functional lifetime due to ODR [Klein Tank et al., 2015, Bessembinder et al., 2023]. Hydraulic load levels are calculated for several representative locations along the Waal for the years 2023, 2050, and 2100, using the 2006 W+ and G climate scenarios.

The water level differences due to ODR are incorporated externally by adjusting the HBN values, as Hydra-NL does not allow manual modification of water levels when calculating HBN values under future climate scenarios. The boundary conditions used in Hydra-NL are based on predefined statistical distributions, which are not dynamically alterable within the model [Geerse, 2015]. The validity of externally superimposing the WLD and excluding additional wave characteristics is is retrospectively validated based on results from Section 4.2.1.

In the assessment, for cases where the ULS is exceeded, a hypothetical reinforcement is introduced with the same crest height surplus and subsidence assumptions as the initial design. This enables quantification of the impact of outward reinforcement on the functional lifetime of future interventions. Soil-based dikes are assigned a functional lifetime of 50 years, while construction-based interventions (e.g. sheet piles) are assumed to last 100 years.

To estimate HBN values for intermediate and future years, a linear interpolation and extrapolation method is applied. Hydra-NL provides HBN values only for the years 2023, 2050, and 2100. Therefore, linear interpolation is used to obtain values for the intervening years. This approach aligns with the Ontwerpinstrumentarium 2014 (OI2014) and is recommended in such cases [Smale, 2018]. For values beyond 2100, the same linear trend is extrapolated, based on the 2050–2100 interval. A more advanced regression method is not feasible due to the limited number of data points and the complex statistical nature of the HBN values produced by Hydra-NL. These values are derived from extensive boundary condition datasets with intricate dependencies and confidence structures. This makes it challenging to assign meaningful statistical weights for a weighted fitting. Therefore, projections beyond 2100 should be interpreted with caution, and no definitive conclusions are drawn regarding the timing or necessity of a second reinforcement cycle.

Assumptions governing the FLR analysis
The FLR analyses is based on a dike section near Oosterhout, with associated HBN values at river kilometre WL_888 (See Figure 3.11 in Subsection 3.2.2). To assess the impact of ODR on functional lifetime, the maximum WLD of 22.6 mm in the Waal is superimposed on the HBN values associated with this location. It is assumed that the dikes are installed in 2025, as HBN values before 2023 are unavailable. For the reference case, this means that the dike is designed for an HBN of 15.00 m +NAP in 2075 for a soil-based body, or for an HBN of 15.27 m +NAP in 2125 for a construction-based dike. Following the assumptions in Section 2.1, a preloaded dike is considered, with a height surplus of 25 centimetres and thus linear subsidence rate of 5 mm/year if subsidence is considered.

In analyses where sheet piles are integrated, the required sheet pile length is incrementally determined with millimetre precision to satisfy the limit state exactly at the end of the original design lifetime. This reference scenario assumes no WLD is imposed yet. In some analyses, this length will subsequently be rounded-up to half metres and the focus then shifts to the functional lifetime shortage.
The calculated length remains fixed throughout the entire FLR assessment. As a result, any increase in HBN values, such as the externally imposed WLD resulting from ODR, reduces the available safety margin and may lead to premature failure. Although the required sheet pile length is formally governed by the DWL, the HBN value from Hydra-NL is used to ensure consistency with the broader FLR analysis. Given that the HBN trend closely follows that of the DWL, this approximation is considered acceptable.

The sheet pile calculations are based on the heave mechanism and are elaborated in Appendix E.3.1. A seepage length of 55.4 metres, associated with the original dike at Ochten (see Section 2.1), is applied. The combination of this seepage length and the projected HBN value results in an exact sheet pile length of 1.0740 metres, which in some analyses is conservatively rounded up to 1.5 metres. The FLR analysis is subsequently evaluated for each year of the dike's lifetime using this fixed sheet pile length. No material degradation is considered.

For the sensitivity analyses, more extreme ranges of influential variables will be considered, including values representative of locations other than Ochten. While the sensitivity of the FLR, and thus the influence of specific parameters, varies per failure mechanism, this will be elaborated in the sensitivity analysis itself.

4.2. Results

4.2.1. Failure mechanism analysis as a basis for ODR feasibility constraints

Overflow and overtopping
The overflow mechanism causes affected dikes to no longer meet safety requirements at the end of their designed functional lifetime. Consequently, the implementation of ODR results in a reduction of the functional lifetime for these dikes. Applying ODR along the entire Waal River increases the design water level by 22.6

mm, which exceeds the crest height used in the original design of affected dikes. As any overflow is classified as failure, the overflow criterion is no longer satisfied. This failure occurs at the end of the functional lifetime, when the most subsidence has taken place and the surplus crest height has diminished. Therefore, it does not constitute immediate failure. The remaining functional lifetime, in combination with the magnitude of WLD, determines the threshold at which the overflow criterion is violated. The extent of FLR and its sensitivity to WLD are discussed in Section 4.2.2.

Overtopping can be confidently considered negligible in the context of a couple of centimetres WLD due to the maximum ODR, and therefore does not result in failure or FLR. Figure 4.2 shows the impact of a 20-metre ODR on the required freeboard height for the overtopping criterion, for a range of floodplain widths. The figure demonstrates that outward expansion reduces the required freeboard height, i.e. a negative additional height, for fetch lengths up to 3.5 kilometres, regardless of average floodplain width and associated water level increase (see Figure 3.15). This indicates that fetch reduction by implementing ODR outweighs the effect of water level rise, and thus ODR does not influence the performance of affected dikes for these fetch lengths. This finding aligns with Camarena Calderon et al. [2016], who observed that wave characteristics are largely unaffected by depths beyond 2 metres.

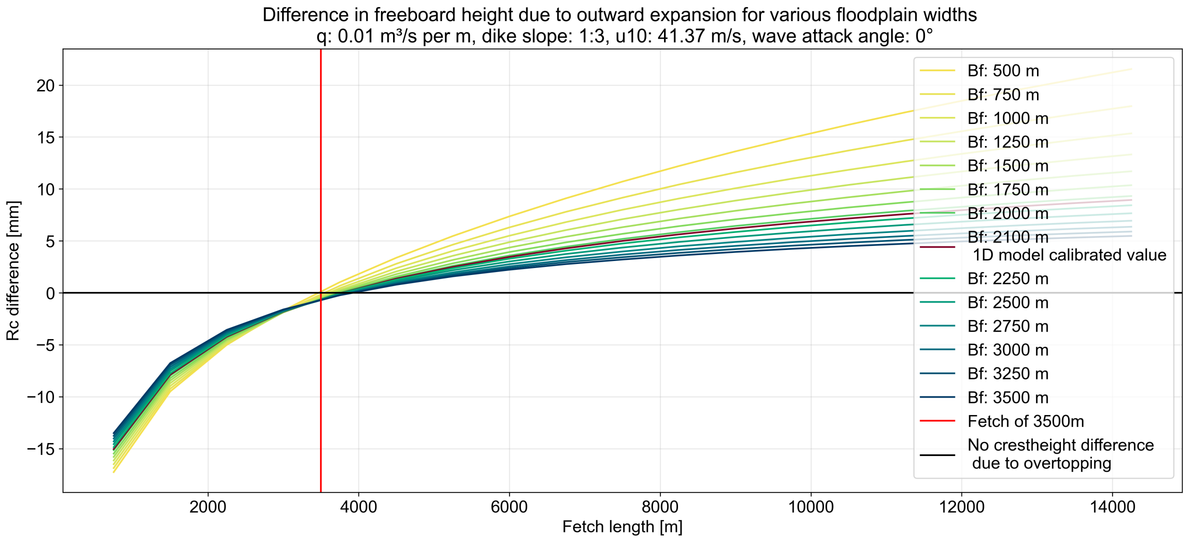

Figure 4.2: Required additional freeboard height (R_c) compared to the original design, for a fixed overtopping rate resulting from WLD due to ODR. Each line colour represents a floodplain width with varying sensitivity to ODR (see Section 3.2.1). Conservative and typical parameter values are used in the overtopping calculations, as indicated in the graph title.

Longer fetches than 3.5 km are affected by ODR, but while theoretically the combination of maximum wind speed, perpendicular wave attack, and long fetch is possible, it is highly improbable. Therefore, an additional sensitivity analysis is conducted where these conservative variables are separated, and the likelihood of simultaneous occurrence of these parameters is discussed in Appendix E.1.1. Based on this sensitivity analysis, it is concluded that the variables influencing overtopping separately have an even less pronounced effect on the freeboard height due to the imposed WLD. Furthermore, the additional crest height due to overtopping would realistically rarely exceed a few millimetres, while the associated water level differences are in the range of centimetres.

Therefore, the only relevant mechanism influencing the required HBN is the water level difference associated with the overflow criterion. The WLD due to ODR can be directly superimposed onto the existing HBN, due to the insignificance of wave and overtopping effects, resulting in the new required HBN value for assessing affected dikes under outward expansion, as already introduced in Section 4.1.2.

Stability

Based on input from C. Spoorenberg (personal communication, May 2025), additional stability measures, such as berms or sheet piles, are not required if the water level increase remains below 10 centimetres. In some cases, even an increase of up to 20 centimetres may be acceptable. The 10-centimetre threshold therefore serves as a direct limit for the application of outward dike reinforcement.

Given that the maximum ODR scenario results in a water level increase of only 2.3 centimetres during peak discharge, it can be concluded that the stability of affected dikes remains uncompromised. No additional constraints or functional lifetime reduction occur, provided that the dikes were originally designed sufficiently to meet the safety standards.

Backward internal erosion (piping)

The effect of the ODR-induced WLD only slightly affects the internal erosion sub-mechanism heave, but this can be critical in cases where a dike barely meets the limit state. Figure 4.3 shows that the heave limit state decreases only slightly with small increases in water level. Although minor, this reduction proves critical for the Tuimeldijk and the construction-based variant with sheet pile length dimensioned precisely to meet the heave criterion. Both fail around the WLD of 22 mm determined for the Waal River (Section 3.2.3). This highlights that even small water level increases can lead to failure in designs that are just sufficient under current conditions. In contrast, designs with more robustness, such as those using rounded-up sheet pile lengths due to practical considerations or the 20-metre ODR, remain unaffected by small variations in water level. Note that although the figure shows WLD values up to 100 mm, it was found that realistic values are limited to approximately 40 mm (Section 3.2).

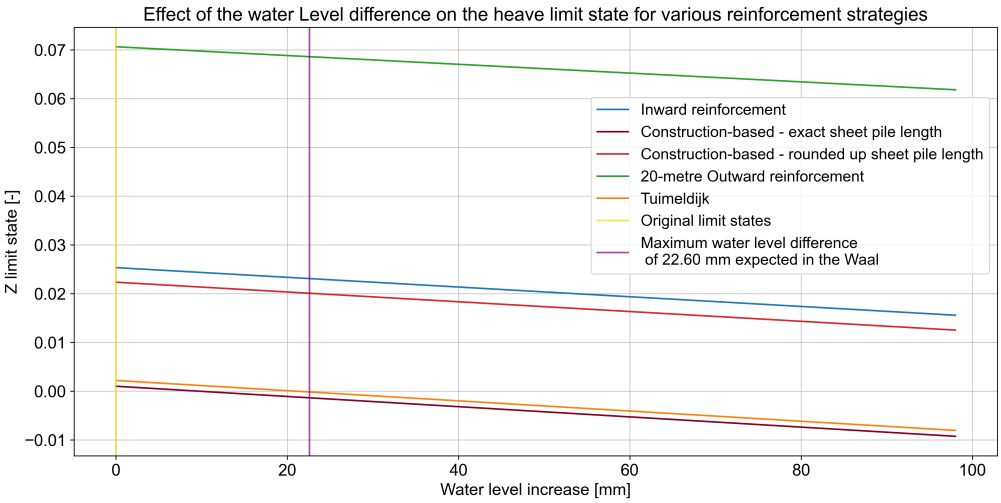

Figure 4.3: Decrease in heave limit state due to WLD imposed by 20-metre riverward expansion. Each line represents the limit state of a conceptual dike design. For the construction-based alternative, sheet pile lengths are analysed using either millimetre precision or rounding to half-metre increments.

The sensitivity of the heave limit state to WLD is influenced by the seepage length, with dikes with longer seepage lengths being less sensitive to WLD and thus showing smaller limit state reductions, as shown in Figure 4.4. This is expected, since longer seepage lengths result in greater damping of the potential piezometric head at the assumed exit point, leading to a lower critical gradient (see Appendix E.3). For all configurations, the effect remains minor and approximately linear, with a reduction of 0.001 per centimetre WLD for the smallest seepage length, decreasing further for wider configurations. Note that the seepage lengths shown in the figure do not directly correspond to the four conceptual designs, but represent a broader range of configurations.

In conclusion, water level differences resulting from downstream ODR only affect the internal erosion mechanism in designs that are marginally dimensioned. Although the effect is limited, it may lead to failure near the end of the functional lifetime in specific cases, though not necessarily immediate failure. The extent of FLR for the heave criterion and its sensitivity are discussed in Section 4.2.2.

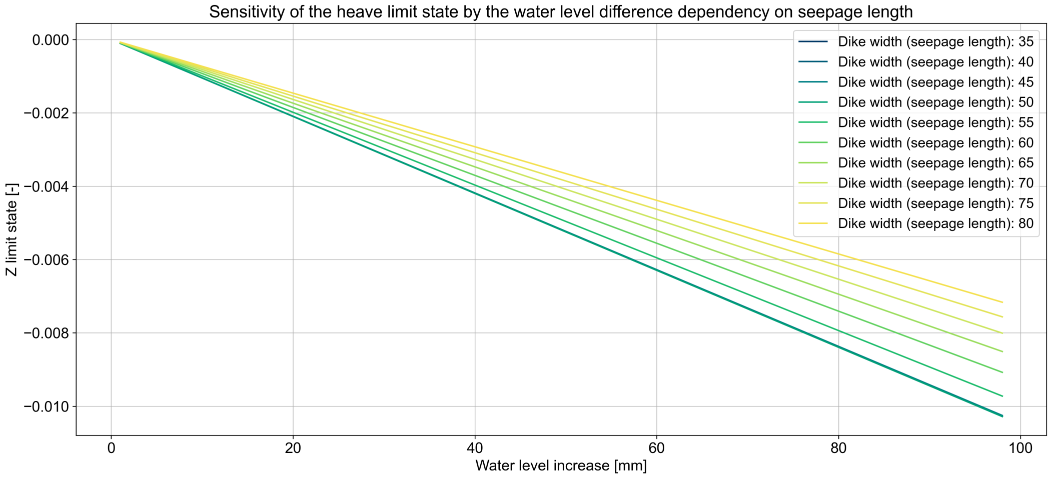

Figure 4.4: Sensitivity analysis of seepage length effects on the reduction of the heave limit state due to water level differences, relative to the designed limit state.

4.2.2. Functional lifetime reduction analysis

As determined in Section 4.2.1, the designed crest height may no longer satisfy the overflow mechanism due to ODR implementation. In some cases, the internal erosion mechanism, governed by heave, may also become insufficient. This section evaluates the resulting FLR and examines the sensitivity of each affected mechanism to influencing factors.

FLR resulting from the overflow sub-mechanism

The functional lifetime reduction of affected dikes along the Waal, resulting from the maximum 22.6 mm increase in water level due to ODR implementation, is 2.2 years based on the overflow sub-mechanism. This is illustrated in the left graph of Figure 4.5, where the characteristic sawtooth pattern is shifted forward due to the increased HBN corresponding to the required ULS (red line). For subsequent reinforcements, no additional lifetime reduction occurs, as these new designs can account for the elevated HBN resulting from ODR.

The annual probability of flooding is not affected in magnitude or behaviour, but is merely shifted to earlier years. This is shown in the right graph, where the crest height of the dike is translated into an annual probability of flooding. As the water level increases, the same probability thresholds (i.e. 1/10,000 per year) are reached earlier in time, even though the overall probability curve remains unchanged. This shifting effect becomes relevant even for small increases in water level.

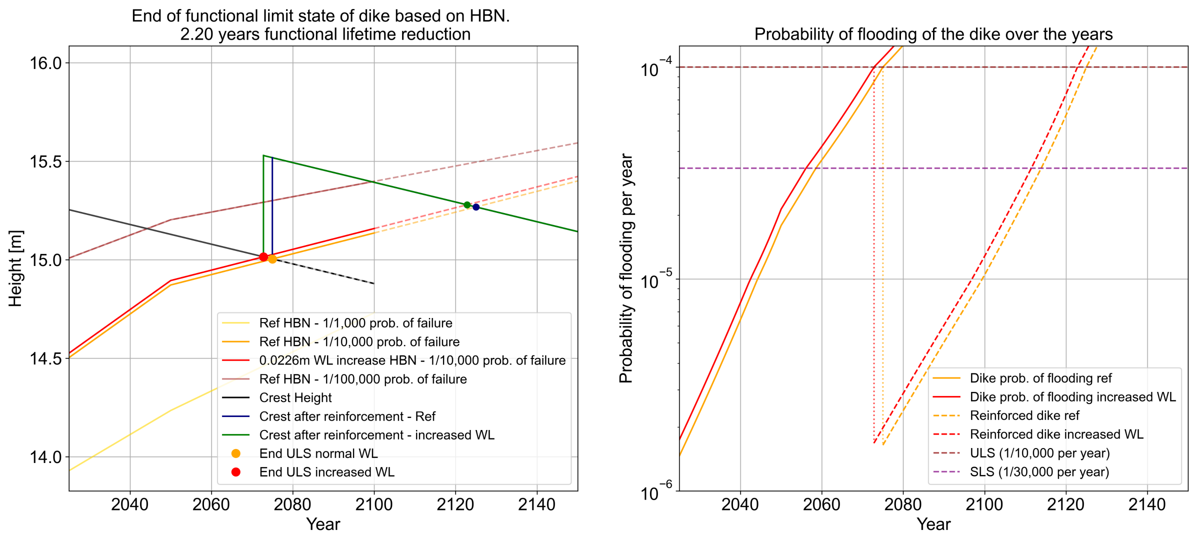

Figure 4.5: **Left:** FLR due to a 22.6 mm water level difference (WLD) imposed by ODR downstream. Black lines show degrading crest height (resistance); red and orange lines represent the new and original HBN for the ULS. Yellow and brown lines indicate required crest heights for alternative ULS conditions. **Right:** Crest height translated into annual flooding probability, with ULS thresholds shown.

Sensitivity Analysis

The reduction in functional lifetime for the overflow sub-mechanism is primarily governed by the projected climate-change-induced increase in HBN values and the superimposed WLD, for a constant annual flood probability (indicated by the red and orange lines), as well as the annual subsidence rate of the dike body (illustrated by the black line), as shown in Figure 4.5.

However, HBN projections, WLD, and subsidence rates vary spatially, and the predicted HBN values depend on the selected climate scenario, as discussed in Section 4.1.2. For the same dike segment near Oosterhout, different assumptions, such as applying climate scenario G or assuming no subsidence, can lead to significantly different lifetime reductions.

To assess the sensitivity of the ODR-induced FLR for the overflow sub-mechanism, hydraulic load levels are also computed for additional locations along the Waal: Wely, Dodewaard, Bonegraaf, Ochten, and Tiel. These represent a range of expected HBN values and are combined with a hypothetical, more extreme range of annual HBN increases, subsidence rates, and water level differences up to the stability limit of 10 cm, as approximated using the 1D model in Subsection 3.2.5.

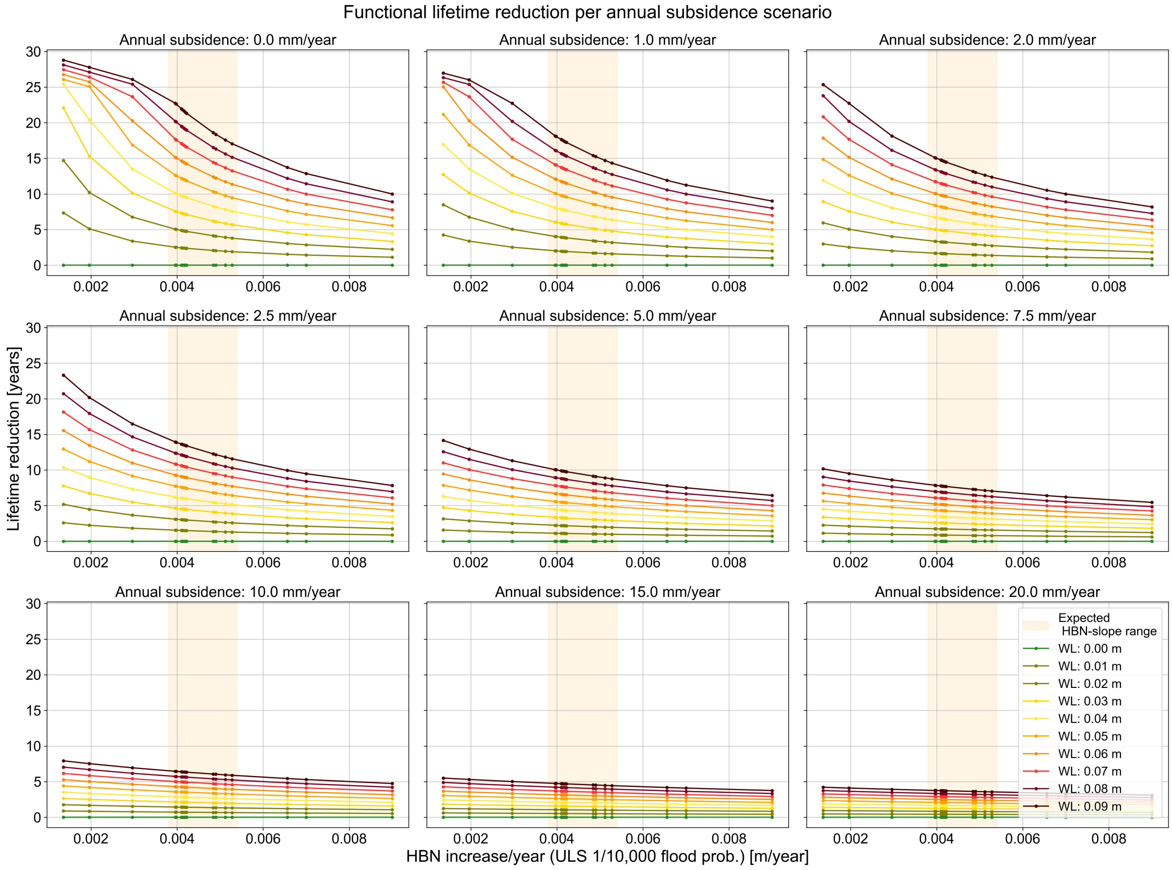

Figure 4.6: Sensitivity analysis of the FLR based on the overflow limit state as influenced by water level differences, subsidence rates, and annual HBN increases under different climate scenarios

Functional lifetime reduction is most pronounced under low subsidence rates, modest HBN increases, and high water level differences, as shown in Figure 4.6. In scenarios with high subsidence or rapidly rising HBN values, the resistance (R) of the reference case (without water level increase) increases steeply relative to the load (S), resulting in a smaller difference between the reference and adjusted cases. Conversely, when subsidence is minimal or HBN rise is limited, the reference case maintains a greater margin, making the additional water level rise more impactful (see also Figure 4.5).

Notably, subsidence has a more substantial influence on lifetime reduction than projected HBN increases. Similarly, larger water level differences lead to more significant reductions, as these accelerate the exceedance of the crest height limit state.

However, it should be noted that scenarios involving either low or non-existent subsidence, or high ODR-induced WLD, are not considered the most plausible, as discussed in Section 2.1 and Section 3.2.5, respectively. Therefore, although such cases are included in the sensitivity analysis, the resulting FLR should not be expected in practice. If subsidence rates were indeed that low, the original functional lifetime of the affected dikes would also be significantly greater due to the presence of the surplus height.

FLR resulting from the heave sub-mechanism

The functional lifetime reduction resulting from the heave sub-mechanism, due to the maximum 22.6 millimetre ODR-induced WLD in the Waal, amounts to 4.3 years. This occurs when the sheet pile length is calculated to precisely meet the heave criterion at the end of the original design lifetime. As shown in the left panel of Figure 4.7, the superimposed increase in hydraulic load causes a downward shift in the limit state curve, leading to earlier exceedance and thus the shortened functional lifetime.

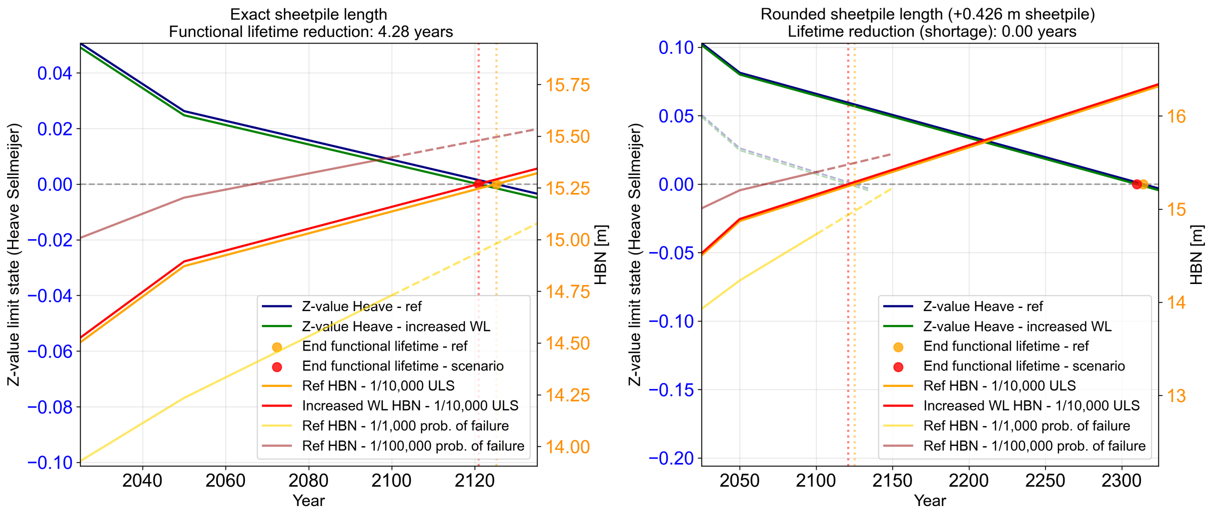

Figure 4.7: **Left:** FLR based on the heave criterion, due to a 22.6 mm WLD from 20-metre ODR downstream. Blue lines show the original heave limit state for a fixed sheet pile length; green lines show the same limit state under increased HBN (red). **Right:** Same analysis with rounded-up sheet pile lengths, showing no FLR.

For sufficiently robust dikes, however, the additional hydraulic load imposed by ODR does not lead to a reduction in functional lifetime. This is illustrated in the right panel of Figure 4.7, where the design is analysed using a rounded-up sheet pile length, reflecting common engineering practice. In this scenario, the limit state remains safely above the critical threshold, and the heave criterion is satisfied for over 200 years.

Sensitivity analysis

Under the assumption of practical rounding, the key parameters governing functional lifetime reduction under the heave criterion are the difference between the exact required and implemented sheet pile lengths, the magnitude of the water level difference, and the slope of the climate-change-induced HBN trend. Subsidence does not affect the heave limit state. The observed linear relationship between the heave limit state and rising HBN may be attributed to the annual interpolation method; alternative HBN trends could yield different outcomes.

To explore FLR sensitivity for the heave limit state, a series of scenarios is assessed using the same method as applied in the overflow and overtopping sensitivity analysis. Heave-related variables are kept constant, as defined in Appendix E.3.
The greatest reduction in functional lifetime occurs under conditions of high water level differences combined with relatively small HBN increases, as shown in Figure 4.8, consistent with the findings from the overtopping sensitivity analysis.

Given the number of variables influencing the heave criterion (see Table E.3), additional sensitivity analyses are performed to evaluate the impact on the heave limit state and resulting functional lifetime, as illustrated in Figure 4.9

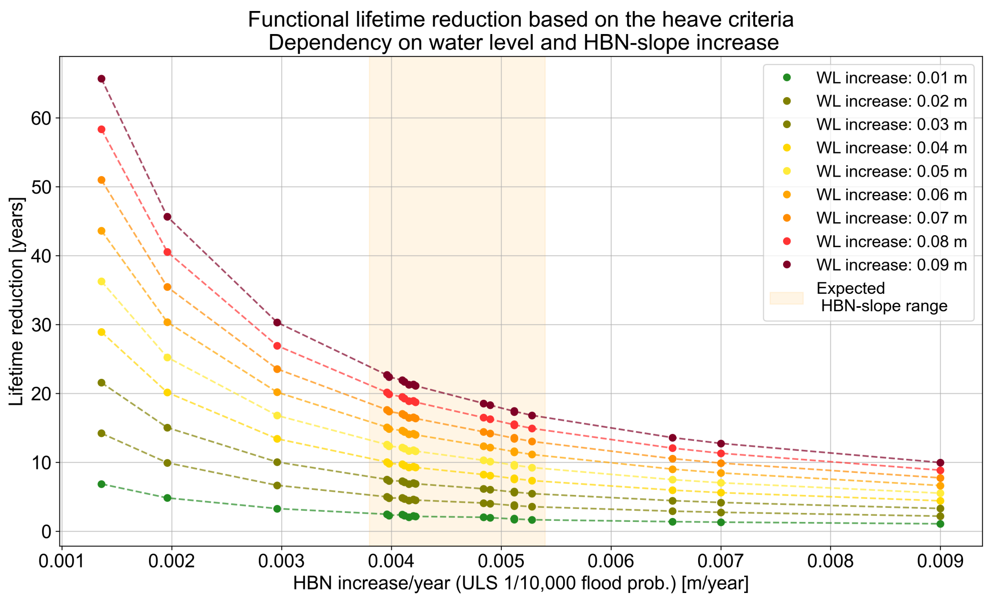

Figure 4.8: Sensitivity of the FLR based on the heave limit state, showing dependency on water level difference and the slope of climate-change-induced annual HBN increase.

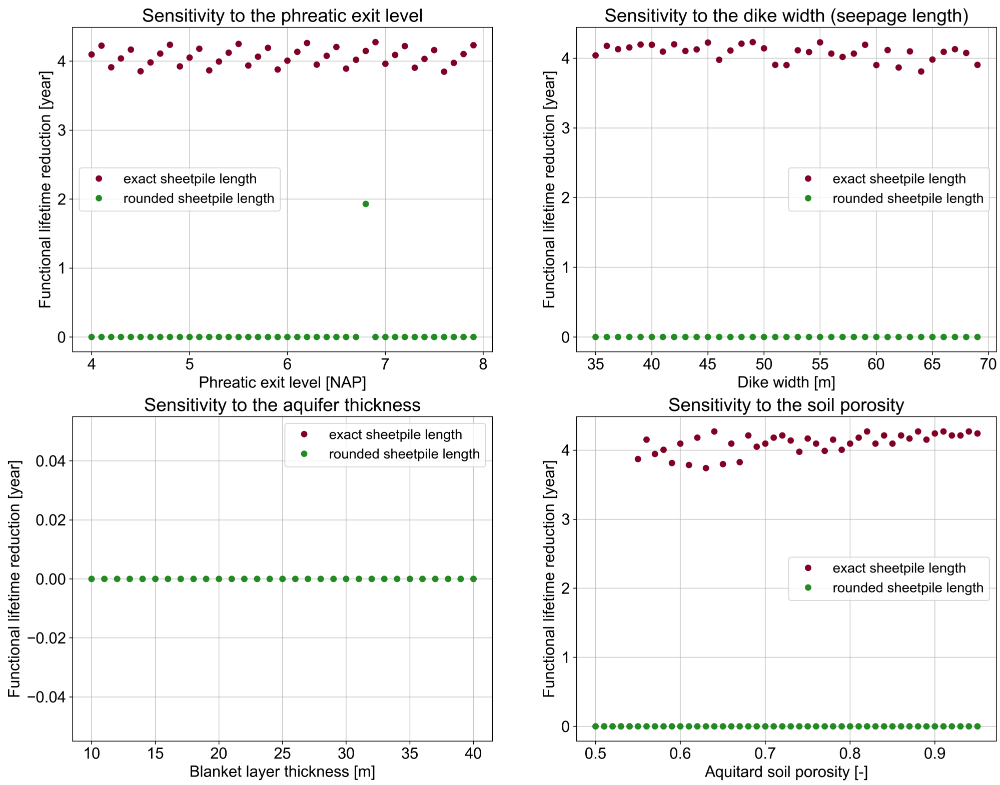

Figure 4.9: Sensitivity of various variables influencing the functional lifetime reduction based on the heave limit, with exact or practically rounded sheet piles

Figure 4.9 shows that variations in heave parameters slightly affect the extent of functional lifetime reduction, but remain close to the previously observed 4.3 years. Although not shown, similar results were found for adjustments in foreshore and hinterland seepage lengths and blanket layer thickness.

In nearly all cases, rounded sheet pile lengths result in no functional lifetime reduction, except for specific combinations of parameters and head differences, as previously observed in Figure 4.9. This confirms that the actual reduction is primarily governed by the robustness of the implemented sheet pile length or the initial resistance provided by the blanket layer.

To quantify this relationship, a scenario is constructed in which the difference between the exact and rounded sheet pile lengths is minimal, and FLR occurs only for the exact case. This scenario involves a phreatic exit level of 5 m +NAP, a dike width of 46 metres, and installation years ranging from 2023 to 2048, during which the HBN incrementally approaches the threshold where both lengths converge.
The analysis is based on HBN values derived for Oosterhout under the W+ climate scenario. In addition, the full range of feasible water level differences identified with the 1D model for the Waal is evaluated, as illustrated in Figure 4.10.

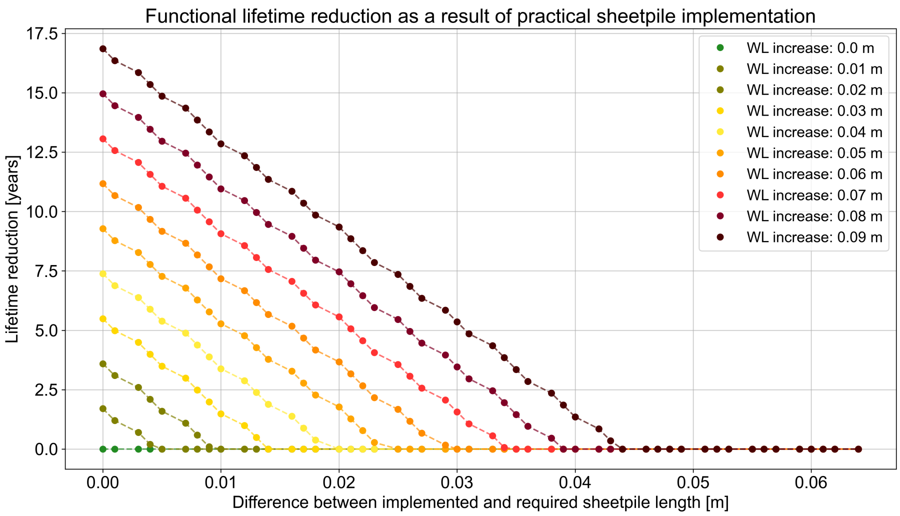

Figure 4.10: Reduction in functional lifetime under the heave criterion, determined by the difference between the required and implemented sheet pile lengths.

If the installed sheet pile includes an additional buffer of more than half the magnitude of the water level increase, no functional lifetime reduction is expected, as illustrated in Figure 4.10. This aligns with the physical interpretation: such a surplus effectively doubles the vertical seepage length increase caused by the water level rise, preventing the hydraulic gradient from reaching critical values for piping. Similarly, if the blanket layer includes a buffer equal to the expected WLD, the same conclusion can be drawn.

Under maximum feasible and conservative conditions of outward expansion in the Waal River, based on the 1D model (4 centimetres; see Section 3.2.5), a surplus of more than 2 centimetres in sheet pile length is sufficient to prevent a reduction in functional lifetime.

Although this cannot be definitively proven, it is reasonable to assume that such conservative rounding is applied in dike design practice along the Waal, due to practical implementation considerations and the tendency to apply robustness in design when time constraints prevent thorough optimisation [Hoogwaterbeschermingsprogramma, 2024a]. Consequently, outward expansion is not expected to result in a reduction of functional lifetime under the internal erosion failure mechanism.

4.2.3. Mitigation strategies for affected dikes under ODR impact

Since the water level response and the resulting reduction in the functional lifespan of affected dikes are identified, various strategies can be considered to mitigate these impacts. This section briefly examines two approaches to mitigate water level differences caused by outward dike reinforcement. The effectiveness of each strategy will be assessed based on associated costs and is further discussed in Chapter 5.

Compensating the missing crest height

The first strategy involves implementing mitigation measures within the current reinforcement project to prevent a reduction in the functional lifetime of affected dikes. Specifically, this is achieved by adding an additional asphalt layer to the crest to compensate for the increased hydraulic load resulting from outward expansion, if lifetime reduction is not accepted. This requires raising the crest height of affected dikes by the same amount as the water level increase, over the complete adaptation length (see Subsection 3.2.5).

Asphalt is watertight and structurally suitable for application on dike crests. Layers can be added repeatedly without compromising stability, as a 10-centimetre asphalt layer does not affect the underlying foundation (e.g., sand-cement mix). Moreover, such water level differences are considered unrealistic and since this also lies below the upper limit based on the stability failure mechanism (see Subsection 4.2.1), the strategy is always applicable. For small elevations (≤ 3 centimetres), only a wearing course is needed. For higher elevations, a binder course (4–5 centimetres) is added beneath additional wearing courses (R. van Haandel, personal communication, July 2025).
The layer can be added either during the outward dike reinforcement project or later, during scheduled maintenance. Asphalt typically lasts 20 years, after which maintenance is required (R. van Haandel, personal communication, July 2025). Coordination with asphalt maintenance schedules is essential to ensure the additional layer is applied before the dike's functional lifetime ends.

Notably, this strategy is also applicable to dikes not affected by ODR. If a dike only exhibits a crest height deficiency of less than 10 centimetres, and no additional stability or piping issues are present, an asphalt layer can be added to extend its functional lifetime. In such cases, ODR still results in a reduction compared to a reference scenario in which all dikes receive the additional asphalt layer.
Furthermore, this strategy requires policy clarity on how strictly the end of the adaptation length is defined (see Subsection 3.2.5).

For the heave criterion, mitigation is generally unnecessary due to the robustness of current designs, as discussed previously in this Section.

Accepting functional lifetime reduction

The second strategy involves accepting a reduction in functional lifetime. This means that upstream dike segments may need to be reinforced earlier than initially planned.
Because each dike responds differently to the same water level difference, this strategy requires systematic collection and maintenance of data for existing dikes. As such, it must be embedded within a broader programme, such as the Flood Protection Programme.

Dikes are designed to meet safety standards throughout their entire intended lifetime. As discussed in Section 4.2.2, increased water levels due to riverwards expansion shift the point at which the ultimate limit state (ULS) is exceeded closer to the present. However, this does not compromise safety before that point. The dike remains compliant with flood probability requirements, and therefore, this loss of functional lifetime can be accepted. Earlier reinforcement entails higher net present costs, introducing additional financial burdens attributable to the outward expansion.

This strategy is only viable if it does not result in temporary non-compliance with ULS criteria. When a dike is nearing the end of its design lifetime and early-stage design and decision-making for new reinforcement has already begun, even a modest reduction (e.g., two years) may constrain the intervention window. This can lead to overly conservative measures, already observed under time pressure in the Flood Protection Programme [Hoogwaterbeschermingsprogramma, 2023], or delayed reinforcement, both of which carry cost and safety implications.

4.3. Synthesis of feasibility implications for ODR

Outward dike reinforcement is technically feasible considering the performance of affected dikes. Of the main failure mechanisms, only the sub-mechanisms overflow and heave of the affected dikes are influenced by the WLD induced by ODR. Hydraulic changes due to ODR have a negligible effect on overtopping, stability is realistically never affected by the ODR-induced WLD, and the other piping sub-mechanisms are not governing.

Based on the overflow mechanism, an FLR of 2.2 years is expected for a 22.6 mm WLD caused by ODR. FLR is primarily governed by the annual climate-change-induced increase in hydraulic load level, the magnitude of WLD, dike subsidence (for overflow), and the additional robustness provided by the blanket layer or sheet pile (for piping).
If the affected dikes exhibit only limited robustness, no FLR due to the heave mechanism is anticipated. For the Waal, a sheet pile length surplus of half the expected WLD is sufficient. This assumption is deemed realistic and therefore, FLR due to heave mechanisms is not governing.

In perspective, the considered FLR may be regarded as relatively insignificant when placed in the broader context of dike management. Historically, reinforcement projects have been required approximately every 30 to 40 years [Van Heezik, 2006], often before the intended functional lifetime is reached. This raises the question of how meaningful a 2-year reduction truly is, also compared to other dominant uncertainties in dike performance, such as the soil variability or the uncertainty in DWL [Warmink et al., 2013].

Still, if the FLR should be addressed, two strategies are proposed. One option is to accept the lifetime reduction. However, to accurately determine the FLR of all affected dikes, the adaptation length must be defined. Furthermore, soil characteristics (i.e., subsidence), installation dates, and reinforcement types should also be properly collected and managed.
If this is infeasible or undesirable, applying an asphalt layer to the crest of affected dikes offers a more practical solution. With the stability limit of 10 cm, this strategy is also suitable for dikes that fail due to overflow and overtopping at the end of their lifetime, even if these are not affected by ODR. Therefore, this strategy is not exclusively applicable to ODR-related cases.

5

Cost-effectiveness assessment

5.1. Approach

This chapter evaluates the cost-effectiveness of outward dike reinforcement (ODR) compared to non-outward alternatives, both in the short and long term. The analysis is based on the four conceptual dike designs, with the 20-metre ODR and Tuimeldijk delineating the range of possible cost outcomes for outward reinforcement. The chapter addresses the third and final sub-question.

First, the dimensions and assumptions of the four conceptual dike designs are briefly revisited. Combined with an established pricing framework, which includes cost components and pricing assumption, the costs per alternative for a single instalment are quantified for a reference scenario. This analysis solely focuses on direct costs under equal boundary conditions, without considering uncertainties or spatial constraints. It illustrates the required short-term investment per alternative to achieve sufficient safety. The purpose is to provide rough, order-of-magnitude estimates that enable a relative comparison between the alternatives.

Once the direct costs for a single reinforcement are determined under the reference scenario, they are embedded in a life cycle cost (LCC) analysis to convert future expenditures into net present value (NPV) and compare the long-term cost-effectiveness of the four alternatives. This LCC analysis follows the guidelines of the Flood Protection Programme (HWBP). The framework also enables the assessment of how changes in the assumed functional lifetime, adaptability, and chosen mitigation strategies affect overall cost-effectiveness.

To further substantiate these, a sensitivity analysis is conducted for each mitigation strategy. This analysis evaluates cost-effectiveness across different contextual scenarios and variables influencing functional lifetime, including uncertainty ranges derived from the previous chapter. It helps identify under which conditions and scenarios ODR proves to be the most cost-effective alternative.

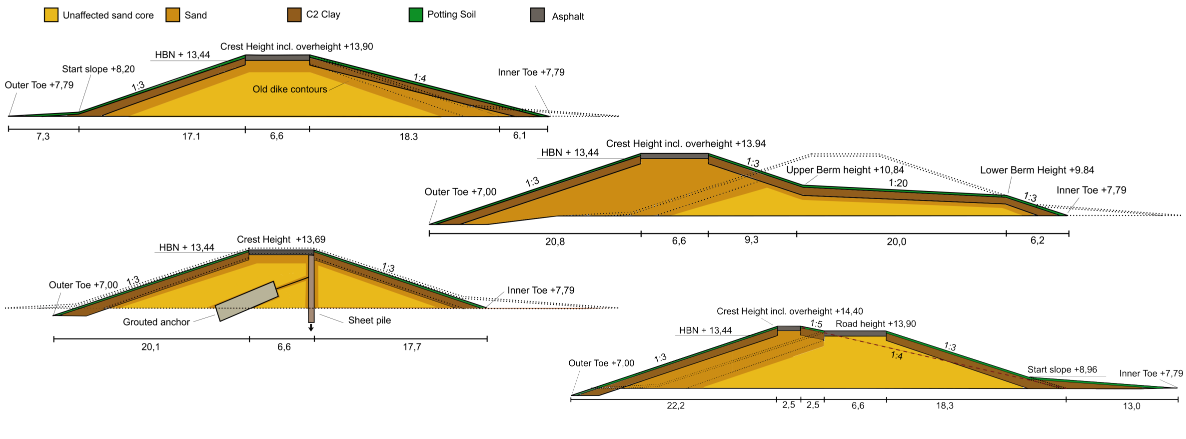

Figure 5.1: Overview of the four conceptual dike designs

5.1.1. Cost estimation framework

The cost estimation is based on the four conceptual dike designs, which are shown again in Figure 5.1. These designs, along with the assumptions that govern each of them, are extensively elaborated in Chapter 2. Their respective dimensions and material compositions are summarised once more in Table 5.1.

Table 5.1: Dimensions of the four conceptual dike designs and associated characteristics

	Inward reinforcement	Construction-based	Outward 20-metre	Tuimeldijk
Land acquisition [m]	6, inward	–	20, outward	5, outward
Sheetpiles	no, possibly for heave	yes, 13.5 m incl. anchors	no, possibly for heave	no, possibly for heave
Total sand volume [m^2]	128	120	157	167
Required new sand volume [m^2]	25	24	119	52
Required new clay volume [m^2]	31	27	42	18
Required new potting soil volume [m^2]	9	8	11	5

The main assumptions underlying the design dimensions and material choices that influence cost are listed below. These are based on the reference cross-section near Ochten (Section 2.1) and hydraulic conditions near the Waal (Appendix E.3). As exact subsurface conditions are unknown, assumptions from multiple projects are combined.

- Stability is assessed using the assumptions in Section 2.1. If not satisfied, sheet piles are required.
  - If buckling is expected due to a stiff Pleistocene sand layer, sheet piles are limited to 5 metres penetration, and anchors are added at 2-metre spacing.
  - Otherwise, sheet pile length equals three times the crest–hinterland level difference.
- For internal erosion, sheet pile lengths are determined following the approach stated in Appendix E.3.1, with seepage length conservatively taken from inner to outer toe.
- Sheet pile calculations are based on a design water level (DWL) of 12.5 m +NAP. At the HBN level of 13.44 m +NAP of the reference cross-sections at Ochten, all designs would require sheet piles under the given assumptions (discussed in Appendix E.3.1).
- Approximately 80% of the original sand core remains intact during reinforcement, unless relocation is required (A. den Teuling, personal communication, May 2025).

The total cost of a dike reinforcement is divided into four main categories: soil cost, land acquisition (if applicable), road works, and sheet pile installation (if applicable). Additional costs related to mitigation strategies are discussed in Sections 5.1.3. The assumptions governing these cost components are as follows:

- Top layers (potting soil, clay, and part of the sand core) are always removed and replaced (A. den Teuling, personal communication, May 2025).
- Soil costs are based on the required new soil volume (Table 5.1) multiplied by unit prices per soil type.
  - New soil is assumed to be sourced from within a 250 km radius of the project site.
  - If recycled soil is used, it is assumed to be fully available from this or a nearby reinforcement project and to meet the required quality standards.
  - In practice, sufficient sand is often available from nearby projects, as it typically does not require strict quality control. However, high-quality clay (C2) is not always readily available. It may not be extractable from the current design (e.g., due to its function as an impermeable layer), or it may no longer meet the required quality criteria. Additionally, the timing between the availability of clay and the need for it is often misaligned [Van Heereveld, 2020].
- Land acquisition is assumed to occur in mutual agreement, without legal proceedings.
- Agricultural land prices are used as a reference for land acquisition.
- If a building must be acquired and demolished, a fixed cost of 1 million euros is assumed per property.

- All designs include a 6.6-metre-wide dike road, including edge closures, consistent with the existing situation and Gastvrije Waaldijk (Appendix C).
- It is assumed that there is an existing road, and this needs to be removed, including foundation and edge closures.
- For the Tuimeldijk, an additional 2.5 metre wide path is included for cycling or walking.

- All sheet piles will be made out of steel.
- If both stability and internal erosion are critical, the governing (longest) sheet pile length is used. If buckling occurs in the latter case, two walls are assumed, one with anchors.
- All sheet piles are rounded to the nearest 0.5 metre as commonly applied in practice by contractors (P. van der Scheer, personal communication, April 2025).
- Sheet piles are installed using static pressing methods due to the assumptions of close proximity to buildings (Appendix C).
- No leakage of the sheet piles is assumed.
- All piles include application cost, finishing and vertical joints.

- It is assumed that all materials are transported by boat over the river and subsequently stored and transported by land close to the site.

To enable a meaningful comparison with other dike reinforcement projects within the Flood Protection Programme, cost data from a comparable, anonymised project by Haskoning is used. These prices are structured according to the Dutch Standard System for Cost Estimation (SSK). The cost comparison is conducted at the baseline scenario ('Nulreferentie') level as defined in the SSK, representing a conceptual design stage without detailed engineering or risk analysis [CROW, 2023]. Table 5.2 shows a price breakdown of the different materials, as provided by Haskoning expert Alex den Teuling (personal communication, April 2025).

The considered costs include materials, storage, transport, labour, machinery, and site-specific requirements, and are based on the assumptions stated previously. The cost estimation includes only capital expenditures related to construction and property acquisition. Engineering costs, project-wide risk contingencies, and miscellaneous supplementary costs are excluded. Furthermore, only the cost categories 'identified direct costs' and 'direct costs to be further detailed' are incorporated into the unit prices. Prices are stated exclusive of VAT (A. den Teuling, personal communcation, April 2025).

Table 5.2: Direct costs of dike materials. The last column represents the cost per linear metre of cross-section along the river axis.

Material	Breakdown	Cost [€]	Unit	Total Cost [€/m]
Sand	Material cost new	16	m^3	24 × sand area [m^2]
	Loading and transport	4	m^3	
	Processing in dike	4	m^3	
	if recycled	10	m^3	
Clay (C2)	Material cost new	20	m^3	33 × clay area [m^2]
	Loading and transport	9	m^3	
	Processing in dike	4	m^3	
	if recycled	26	m^3	
Potting soil	Material cost new	7.5	m^3	15 × potting area [m^2]
	Loading and transport	3.5	m^3	
	Processing incl. sowing	4	m^3	
	if recycled	7	m^3	
Land Acquisition	Purchase agricultural land	11	m^2	11 × expansion [m]
Property Acquisition	Purchase of 1 property	1,000,000	1×	1,000,000
Road removal	Remove asphalt top layer	10	m^2	22 × road width [m]
	Remove/store edge closure	3.5	m	
	Remove and clean foundation	5	m^2	
Asphalt layer	Material + application	32	m^2	32 × road width [m]

Material	Breakdown	Cost [€]	Unit	Total Cost [€/m]
Road construction	Install foundation	7	m^2	69 × road width [m]
	Apply edge closure (2 sides)	15	m	
Steel sheet piles	Material cost	182	m^2	182 × depth [m] + 330
	Application + finishing	150	m	
	Vertical joints between planks	180	m	
	Grout anchor (if applicable)	500	1×	500 / c/c distance [m]

It is important to note that the resulting cost estimates are lower than what would typically be expected in real-world projects. This is intentional, as the alternatives are defined at a conceptual level within the baseline scenario, where detailed cost breakdowns and comprehensive risk assessments are not yet applicable. In practice, dike designs require significantly more detail and are subject to site-specific boundary conditions, which ultimately determine the actual costs and the most cost-effective solution.

Reference scenario

For the main part of the cost-effectiveness analyses, a reference scenario is defined to ensure consistency in cost assumptions. In this scenario:

- If an alternative is considered, it is assumed that only this alternative is implemented across the entire considered reach.
- All dike alternatives are assumed to be constructed entirely with new soil.
- No buildings require demolition on either side of the dike.
- No alternative is prone to backward internal erosion.
- The intervention length is set to 32 kilometres, covering the entire considered Waal reach.

No additional spatial constraints are imposed, and uncertainties in input parameters are not considered at this stage. Any deviation from this reference scenario is explicitly mentioned or addressed through a sensitivity analysis.

5.1.2. Life cycle cost analysis

To assess the long-term cost-effectiveness of the four conceptual alternatives, a life cycle cost (LCC) analysis is conducted using the net present value (NPV) methodology. This methodology accounts for the time value of money, which reflects the principle that money today is worth more than money in the future. This is because funds received earlier can be invested to generate returns, while future funds have lower immediate utility and are subject to greater uncertainty [Shou, 2022]. This principle is incorporated through a discount rate (R), which adjusts annual costs (C) to their present value. Subsequently, reinforcement costs far in the future are more heavily discounted and thus costs less than reinforcement costs closer to the present. Consequently, postponing reinforcement leads to lower present-value costs compared to immediate reinforcement. Figure 5.2 conceptually illustrates how the NPV methodology influences cost distribution over time and highlights the role of a dike's functional lifetime within a fictive reinforcement scenario.

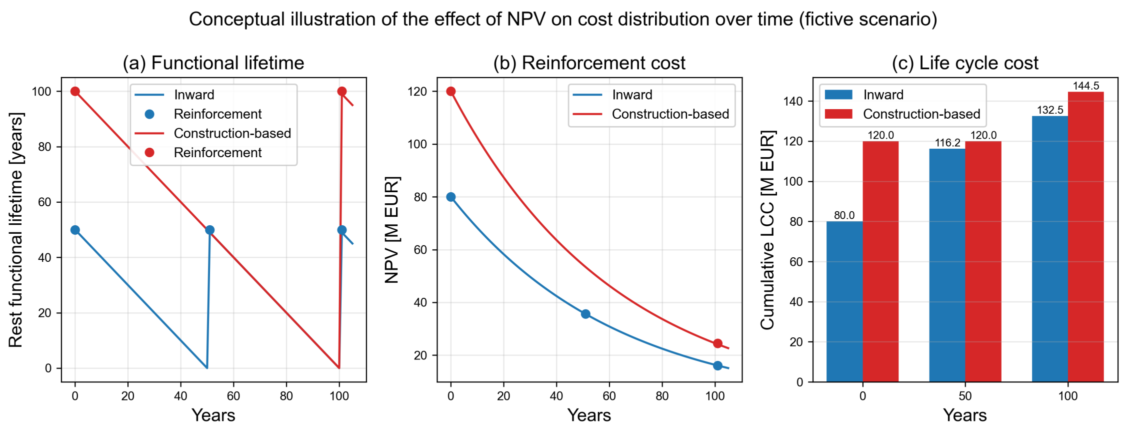

Figure 5.2: Conceptual visualisation of the effect of NPV on cost distribution over time, as part of a LCC analysis. The figure shows how the timing of reinforcement influences the present-value cost in a fictive dike reinforcement scenario.

The NPV is calculated for each year (t) using Equation 5.1, allowing for a comprehensive comparison of total costs over a defined forecast horizon (T). This approach facilitates comparison between alternatives with differing functional lifetimes. The use of LCC with the NPV methodology for comparing dike alternatives is mandated by the HWBP [Rijke and Hertogh, 2014].

$$NPV = \sum_{t=1}^{T} C \times \frac{1}{(1+R)^t} \tag{5.1}$$

This study broadly follows the HWBP methodology for LCC assessments [Hoogwaterbeschermingsprogramma, 2021], with several tailored assumptions:

- A discount rate of R = 1.6% is applied, in line with HWBP standards [Hoogwaterbeschermingsprogramma, 2021], as the costs are fixed to the economic growth.
- A fixed forecast horizon of 100 years is applied.
- Soil-based alternatives are assumed to have a functional lifetime of 50 years, while construction-based alternatives are assumed to last 100 years.
- The LCC includes investment costs at t = 1, followed by annual operation, management, and maintenance costs. At least one reinforcement is assumed within the forecast horizon.
- Annual operation, management, and maintenance costs are set at €8,000 per kilometre of intervention, based on typical values for primary flood defences [Rijke and Hertogh, 2014]. More precise estimates per alternative are difficult to obtain.
- The initial reinforcement is referred to as the instalment, while subsequent reinforcements are termed next reinforcements.
- In HWBP assessments, next reinforcements at the end of the functional lifetime are assumed to incur the same cost as the initial instalment [Hoogwaterbeschermingsprogramma, 2021].
- For the 20-metre outward expansion and Tuimeldijk alternatives, differentiated costs are applied for next reinforcements:
  - For the 20-metre outward expansion, the next reinforcement follows the approach described in Section 2.3, reflecting its adaptability. This variant avoids additional mitigation and acquisition costs and requires significantly less soil than the initial instalment.
  - For the Tuimeldijk, future reinforcements still require new acquisition and mitigation costs, as well as a sheet pile to address internal erosion due to rising HBN levels. This allows assessment of its potential as a low-regret design.

Table 5.3 shows the values applied for the next reinforcement of the 20-metre outward expansion and the Tuimeldijk, as discussed in the list above.

Table 5.3: Dimensions and materials for the two outward dike alternatives for next reinforcements, after the first instalment

	Outward 20-metre next reinforcement	Tuimeldijk next reinforcement
Land acquisition [m]	–	5 metre, outward
Sheetpiles	no	yes, 2.5 m for heave
Total sand volume [m^2]	174	167
Required new sand volume [m^2]	48	52
Required new clay volume [m^2]	43	18
Required new potting soil volume [m^2]	12	5

5.1.3. Quantifying mitigation costs due to hydraulic impact

With the cost assumptions defined over a fixed forecast horizon, only the mitigation strategy for the water level difference (WLD) caused by outward expansion remains to be addressed. As outlined in Section 4.2.3, this can be resolved either by adding an asphalt layer to affected dikes or by accepting a reduction in their functional lifetime (FLR).

Strategy 1: Additional asphalt layer
The first strategy to address the hydraulic impact of outward dike reinforcement is to apply an additional asphalt layer to the affected dikes. In this approach, the reduction of functional lifetime is not accepted. Instead, all affected dikes are elevated to meet the ultimate limit state for overflow and overtopping at the end of their functional lifetime, as discussed in Section 4.2.2. This is visualised by the orange line in Figure 5.3.

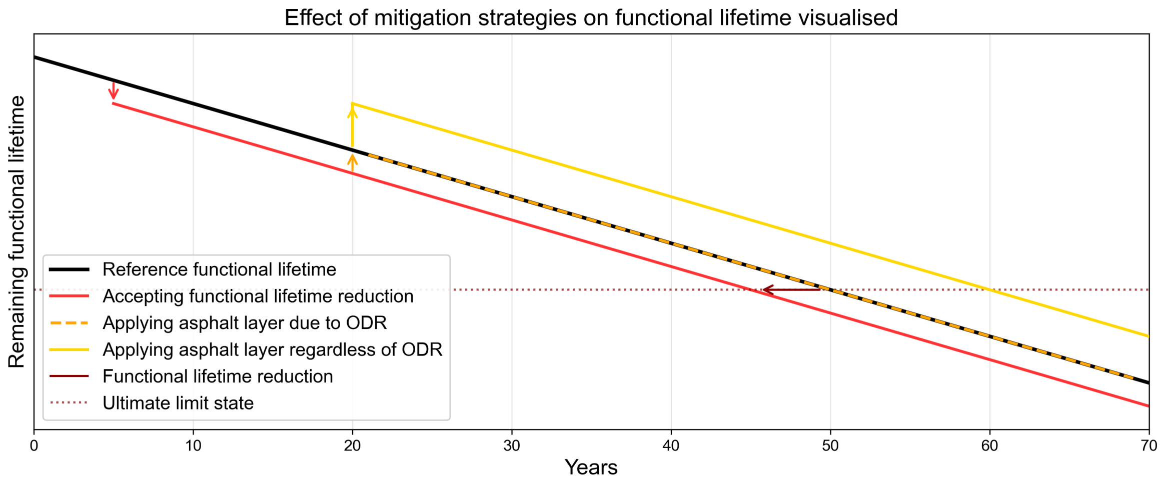

Figure 5.3: Effect of either accepting functional lifetime reduction (FLR) or applying an additional asphalt layer on affected dike, which extends the functional lifetime, as discussed in Subsection 4.2.3.

This strategy is only applicable if it can be assumed that internal erosion does not lead to functional lifetime reduction, i.e. all affected dikes exhibit slight robustness. As determined in Subsection 4.2.1, the other failure mechanism are not affected by ODR implementation.
The cost of mitigating water level differences is added to the outward reinforcement alternatives as mitigation costs, determined as follows:

- It is assumed that the asphalt layer will be applied as part of the ODR reinforcement, rather than during future road maintenance. This assumption is made due to the lack of data on when specific roads on individual dike sections are scheduled for maintenance.
- Water level differences are derived directly from D-Hydro simulations (Section 3.2.3).
- The adaptation length is estimated using the simplified 1D model, so the extent of outward expansion directly influences the magnitude of water level rise and thus the number of affected upstream dikes.
  - Although closely aligned with D-Hydro results, the 1D model is slightly more conservative, resulting in longer adaptation lengths.
  - A threshold of 0.1 mm and a calibration factor of 1 are applied, meaning the model is not calibrated to D-Hydro, which would otherwise yield overly conservative lengths for peak discharges (see Appendix G).
- For simplicity, the maximum WLD is used to determine the required elevation for a reach of affected dikes, despite lower water level differences upstream or downstream of the considered reach.
- All dikes on the opposite bank along the intervention length, as well as all upstream dikes within the adaptation length, are assumed to require mitigation.
- As described in Section 4.2.3, the amount of asphalt required depends on the WLD (R. van Haandel, personal communication, July 2025).
  - For water level differences below 3 cm, a single 3 cm wearing course of AC11 is applied, costing €4 per square metre.
  - For differences between 3–8 cm, both an AC11 wearing course and a binder course of AC16 are required. This binder course costs €10 per square metre.
  - For differences above 8 cm (up to the 10 cm stability limit), one wearing course and two binder courses are applied.
  - If mitigation is applied, existing edge closures are removed and reapplied (see Table 5.2).
  - Labour, machinery and site-specific requirements are assumed within the asphalt layer prices.

As previously stated in Subsection 4.2.3, this strategy is also applicable to dike reinforcement projects where ODR is not implemented, as it extends the functional lifetime. This is illustrated in yellow in Figure 5.3.
In such cases, the same costs would apply regardless of whether ODR is implemented, effectively neutralising the cost comparison between outward and non-outward alternatives for this strategy and shifting the analysis towards a FLR assessment.
However, for the purpose of this analysis, the strategy is considered only in scenarios where dikes are affected by ODR. The implementation of ODR is benchmarked against the originally intended functional lifetimes of the affected dikes, and this strategy then ensures that these lifetimes can still be achieved. As a result, the impact of ODR becomes directly quantifiable in terms of mitigation costs.

Strategy 2: Accepting functional lifetime reduction
If the second strategy is adopted, the functional lifetime reduction (FLR) of affected dikes is accepted without further mitigation, as described in Subsection 4.2.3 and visualised by the red line in Figure 5.3.
Accepting FLR implies that affected dikes will require reinforcement earlier than originally planned. This shift accelerates expenditures that would otherwise occur later in the planning horizon, thereby increasing the net present cost, as illustrated in the middle plot of Figure 5.2. Although the total reinforcement cost remains unchanged, its financial weight increases when expenditures are brought forward in time. These additional present-value costs are directly attributable to the outward expansion alternative and must be considered in the overall cost-effectiveness assessment.

The functional lifetime reduction is governed by multiple variables and is calculated in the same manner as discussed in Section 4.1.2. Therefore, the following requirements are needed.

- Information of the affected dikes
  - The type of upstream reinforcement: soil-based (50-year design life) or construction-based (100-year design life).
  - The installation year of each dike reach, to determine its remaining functional lifetime.
  - The distance from the reinforced dike, which determines the expected water level increase via the adaptation length (see Chapter 3).
  - The length of each reach, to assess whether it falls within the adaptation zone.
- The amount of water level difference at the location of an affected dike.
- Hydraulic load levels at a certain location for the whole future timespan.

If the FLR induced by outward expansion is known, it is subtracted from the remaining lifetime of the affected dikes. This analysis assumes timely reinforcement at the end of each segment's adjusted lifetime, ensuring safety norms are maintained.
To quantify the effect of FLR in costs, the original life cycle cost of the entire affected river system is assessed. The difference between this baseline and the scenario with advanced reinforcements reflects the additional cost attributable to outward expansion, enabling consistent comparison across the mitigation strategies.

Due to the complexity of individual dike segment, which varies in length, type, installation date, and reinforcement strategy, the analysis is simplified to enable a first-order cost estimation.

- The river system is divided into dike reaches, assumed to correspond to a single reinforcement project. Where data is available, actual reaches and installation dates are used.
- If installation dates are unknown, the year 2023 is assumed. From this point onward, HBN values are known and applied in the FLR assessments.
- If the reach length is unknown, a default value of 15 km is applied.
- Each reach is assumed to consist of 50% inward soil-based reinforcement and 50% construction-based reinforcement.
- If both backward erosion and overtopping contribute to functional lifetime reduction (FLR), the most critical mechanism governs.
  - In some cases, it is assumed for simplicity that sheet piles are robustly implemented, preventing FLR due to backward erosion (see Section 4.2.2). This assumption is made solely to isolate and quantify the FLR effect from overflow and overtopping, as FLR due to heave is highly location-specific.

- If water level differences result in a reduction of the functional lifetime, this will be rounded up to the nearest whole year.
- An additional lifetime reduction may be imposed to assess the sensitivity of the results to functional lifetime reduction caused by external factors.
- The four conceptual designs are used to assess FLR criteria, rather than the actual designs present in each reach.
  - The costs are scaled to cost per metre.
  - The conceptual inward alternative was initially developed for a DWL 12.5 m +NAP (see Appendix E.3.1). However, actual hydraulic boundary conditions (HBN) used in the analysis are higher, meaning that, according to the criteria, sheet piles would be required. This assumption is made solely to quantify the functional lifetime reduction (FLR) due to backward erosion. Importantly, the cost of these sheet piles is not included in the LCC of the inward alternative.
- For upstream reaches, the water level increase is taken at the downstream end and applied uniformly, as this results in the maximum WLD within the reach.
- For dikes on the opposite bank of ODR, the water level increase is taken at the upstream end and applied uniformly.
- The adaptation length approximation determines whether a reach is affected upstream.
- The same HBN values as used in Section 4.1.2 are applied, provided these approximately align with the start of the reach.
- It is assumed that the soil beneath all dikes is preloaded, with a subsidence rate of 5 mm/year, corresponding to a surplus height of 25 cm (Section 2.1).

Considered ODR implementation sections
This strategy is analysed for two ODR implementation cases within the reference scenario, each characterised by a different intervention length. The first case considers reinforcement only between Dreumel and Boven Leeuwen, allowing the use of actual data for upstream dikes based on previous projects, and avoiding overly hypothetical assumptions.
However, this reach does not cover the full Waal, as analysed in the hydraulic impact assessment. Therefore, a second case considers 20-metre ODR along the entire southern Waal bank. This enables direct application of the hydraulic impact results from Chapter 3, allowing for a consistent comparison between Strategy 2 and Strategy 1. This broader analysis, however, requires more assumptions regarding the affected dikes upstream.

Cost approximation for Dreumel–Boven Leeuwen
Figure 5.4 provides spatial context by visualising the D-BL section and its affected surroundings using a Google Earth view. This is followed by Figure 5.5, which presents a schematic overview of the associated river reaches. It includes the reach lengths and installation dates, derived from the Flood Protection Programme [Hoogwaterbeschermingsprogramma, 2025], as well as the assumed reinforcement type and the estimated water level increases resulting from outward expansion at Dreumel–Boven Leeuwen.

Figure 5.4: Approximate visualisation of the location of affected sections (white and blue) resulting from reinforcement at Dreumel–Boven Leeuwen (red), based on the interactive map of the Flood Protection Programme (HWBP) [Hoogwaterbeschermingsprogramma, 2025]. The lines perpendicular to the river axis indicate the boundaries of the sections considered in Figure 5.5

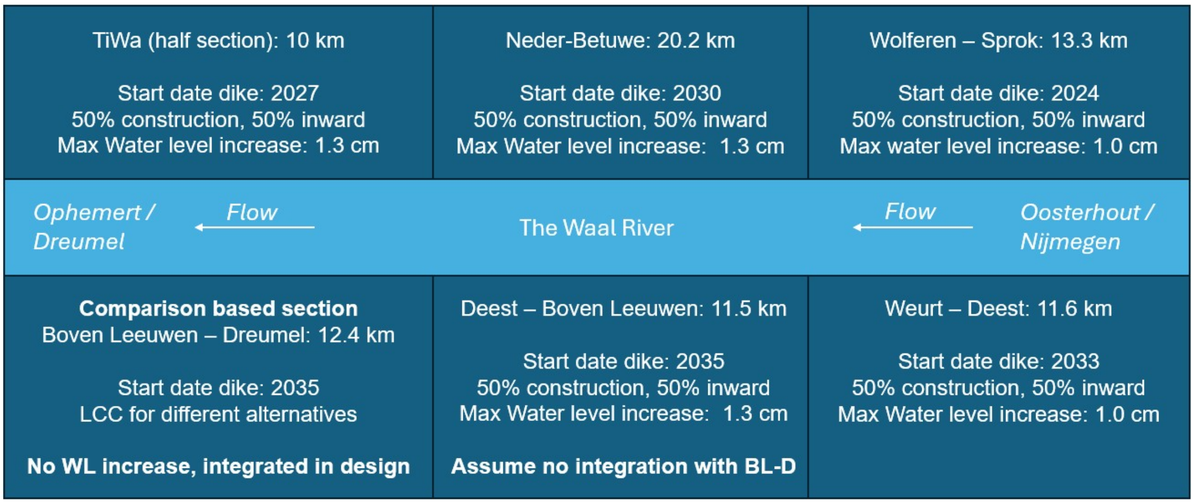

Figure 5.5: Schematisation, data overview and associated WLD of the affected reaches due to ODR implementation at Dreumel-Boven Leeuwen.

This case is selected solely for analytical purposes, as it provides the most comprehensive dataset of affected dikes, and the hydraulic impact for this reach is well understood. There is no connection made between the selected case and the future Flood Protection Programme project of the southern Waal dikes.

The water level differences and corresponding adaptation lengths for each reach and both outward expansion alternatives are derived using Figures 3.14, 3.12, and the 1D model. The following assumptions and results apply:

- **20-metre outward expansion**
  - Maximum water level increase: 13 millimetre, based on Figure 3.12.
  - Adaptation length: 60 km, derived using the simplified 1D model (as in Strategy 1), beyond which the water level difference drops below 0.1 millimetre.
  - Upstream of Wolferen-sprok, assume a remaining water level difference of 0.6 millimetre, based on Figure 3.14.
- **Tuimeldijk alternative**
  - Maximum water level increase: 3 millimetre.
  - Adaptation length: 43 km, based on the same modelling approach.
  - No D-Hydro simulations are available for shorter intervention lengths for smaller expansions; therefore, the same proportional relation between the maximum water level difference and the water level at the start of the reach is applied as for the 20-metre expansion.
  - Estimated water level difference: 2 mm downstream and 1 mm upstream of the Weurt–Deest reach.
- **Reach-specific assumptions**
  - Although the Dreumel-Boven Leeuwen and Boven Leeuwen-Deest sections share the same installation date, the latter is not assumed to be adaptable to the water level differences. Therefore, functional lifetime reduction (FLR) applies.
  - For reaches upstream of Nijmegen, no D-Hydro data is available. Consequently, earlier assumptions are maintained. From Figure 3.14, a starting water level difference of 6 millimetre is assumed.

Cost approximation for the whole Waal River
To assess the costs resulting from accepting FLR for affected upstream dikes, assuming reinforcement of the entire southern Waal banks, simplifications are required for the river dikes upstream of Nijmegen due to the absence of available data. The assumptions are outlined as follows:

- **Reach-specific assumptions**
  - It is assumed that the entire southern bank will be installed in 2035, corresponding to the installation year of the Dreumel–Boven Leeuwen and Boven Leeuwen–Deest sections.
  - The northern banks of the Waal will be assessed using the same approach as applied to the Dreumel–Boven Leeuwen reach, but with updated water level data from Figure 3.12.
  - For reaches upstream of Nijmegen, no D-Hydro data is available. Therefore, previous assumptions are retained.
  - 50% of each reach is assumed to be construction-based, and the other 50% to consist of soil.
  - The HBN values near Oosterhout are assumed for all upstream dike segments.
  - A new reach is assumed every 15 kilometres, each with a distinct WLD resulting in FLR.
  - Although the adaptation lengths fall outside the D-Hydro simulation range, extrapolated adaptation lengths are used to estimate water level differences, as described in Appendix G and shown in Figure G.7.
  - The adaptation lengths extend to the Pannerdensche Kop bifurcation. No additional dikes around the Pannerdensche Kop bifurcation are assumed to require mitigation, except for the affected dikes on both sides. These are treated as if the river continues as a single branch, rather than splitting into two. No further effects on adaptation length are considered.
- **20-metre outward expansion**
  - Maximum water level increase: 22.6 millimetres, based on Figure 3.12.
  - Adaptation length: 67 kilometres, derived using the simplified 1D model (as in Strategy 1), beyond which the WLD drops below 0.1 millimetres. This is only slightly longer than the adaptation length for the D–BL reach.
  - The first 15 kilometres are assumed to experience a water level difference of 22.6 millimetres. After this initial reach, the difference drops to 13 millimetres, then to 6 millimetres after another 15 kilometres, and finally to 2 millimetres for the last 22 kilometres (Figure G.7).
- **Tuimeldijk alternative**
  - Maximum water level increase: 6.7 millimetres.
  - Adaptation length: 50 kilometres, based on the same modelling approach.
  - No D-Hydro simulations are available for shorter intervention lengths with smaller expansions; therefore, the same proportional relation between the maximum water level difference and the water level at the start of the reach is applied as for the 20-metre expansion.
  - Estimated water level difference: 3 millimetres after the first 15 kilometres, 1 millimetre further upstream for the final 20 kilometres.

5.1.4. Method for the sensitivity analysis

The previous subsections outlined the approach used to assess the cost-effectiveness of ODR relative to non-outward reinforcement, based on the reference scenario and assuming fixed functional lifetimes for the long-term analysis. While useful for comparison, this does not fully reflect real-world conditions. Therefore, cost-effectiveness is further evaluated through a sensitivity analysis.

The sensitivity analysis for short-term cost-effectiveness consists of two parts:

- The direct costs between the alternatives are briefly investigated when recycled soil is applied.
- The effect of intervention length on the cost per metre dike is examined, where total reinforcement costs are distributed across a range of intervention lengths.
  - For outward variants, only the additional mitigation costs of an asphalt layer are considered and divided over the intervention length range.
  - For the inward variant, a scenario involving the demolition of 10 houses is sketched, with these additional costs similarly distributed.

The sensitivity analysis for long-term cost-effectiveness is conducted over a 100-year forecast horizon and is divided into two components:

- **Scenario-based sensitivity**, which explores how contextual factors, such as material choice, piping sensitivity, spatial constraints, and intervention length affect the viability of ODR.

- **Functional lifetime related sensitivity**, which examines how changes in functional lifetime and variables influencing the FLR (see Subsection 4.2.2) affect the cost-effectiveness of ODR, particularly when this FLR is accepted.

Scenario-based sensitivity is assessed using Strategy 1 and therefore excludes sensitivities related to potential FLR. The additional scenarios introduce variations in soil characteristics, cost parameters, spatial constraints, and intervention lengths. The following scenarios are considered:

- **Full-scale reinforcement along the Waal River** (entire 32-kilometre reach, representing long intervention lengths)
  - Identical to the reference scenario, but with full application of recycled soil.
  - Subsoil is prone to backward internal erosion, simulated by lowering the phreatic exit depth to 7 m +NAP. Only the 20-metre outward expansion avoids the need for a heave screen; both inward expansion and the Tuimeldijk design require a 2.5-metre heave screen.
  - Demolition of 150 houses on the landward side, representing an upper-bound scenario that includes urban areas. This reflects the maximum number of properties located on the inner toe that would require removal if inward reinforcement were realistically applied along the full reach. The actual number of houses that require demolition may be even higher.
  - Demolition of only 50 houses on the landward side, excluding densely built areas, highlighting that many houses remain affected even when urban centres are left out.
  - Sheet pile costs increase by 25%.
  - To simulate intensifying spatial pressure, property acquisition costs are projected to double by 2050, while land acquisition costs increase fivefold. As before, it is assumed that 50 houses on the landward side must be purchased.
- **Reinforcement over a 15-kilometre reach** (representing typical project-scale intervention lengths)
  - The same scenarios as listed above.
  - Instead of 50, a limit of 30 houses requiring demolition on the landward side is considered, based on the reach between Dreumel and Boven Leeuwen, where fewer houses are located on the inner toe and urban areas have been excluded.
  - A combined scenario involving piping-sensitive soil, demolition of 30 houses, and full recycled soil application is also considered.
- **Local reinforcement over a 5-kilometre reach** (representing short intervention lengths)
  - The same scenarios as for the full Waal River reach, excluding increased sheet pile costs.
  - Similar to the 15-kilometre reach, the combination of the three scenarios and future spatial pressure is evaluated, this time involving only 10 house demolitions.

Functional lifetime related sensitivity is assessed using Strategy 2, as changes in functional lifetime directly translate into cost differences. These scenarios focus on climate-change-induced stressors and their impact on HBN, assumed functional lifetimes and affected dikes types, and subsidence. The following scenarios are considered:

- **Full-scale reinforcement along the Waal River**
  - Annual subsidence across the affected stretch is reduced to 1 millimetre/year.
  - Annual subsidence across the affected stretch is increased to 1 centimetre/year.
  - A low-end climate change scenario, where the annual HBN slope increase is reduced to 0.002 metres/year, representing a climate trajectory significantly milder than the G-climate scenario.
  - A severe climate change scenario, where the annual HBN slope increase is raised to 0.007 metres/year, representing a climate trajectory more extreme than the W+ scenario.
  - Functional lifetime of soil-based designs is extended by 20 years, reflecting higher adaptability.
  - Functional lifetime of construction-based designs is reduced by 20 years, representing potential low-regret rigid solutions.
  - The FLR of construction-based designs is combined with 25% higher steel prices.
  - Functional lifetime of construction-based designs is further reduced by 50 years, reflecting the unforeseen need to replace sheet pile walls due to sudden loss of structural reliability.
  - The functional lifetime of all alternatives is reduced by 20 years to simulate a new reinforcement programme based on updated technical insights.

- All affected dike segments consist solely of soil-based alternatives, which require earlier reinforcement, resulting in a higher net present value.
- All affected dike segments consist solely of construction-based alternatives, which require later reinforcement but incur higher reinforcement costs.
- Combined scenario: reduced HBN increase and 1 millimetre/year subsidence, representing a worst-case for FLR mitigation.
- Combined scenario: increased HBN increase and 20-year lifetime reduction for construction-based designs, simulating intensified climate pressure.

- **Reinforcement over a 12.4-kilometre reach** (Specifically Dreumel-Boven Leeuwen)
  - The same 20-year FLR of all variants and the affected dike variant type scenarios as listed under the full-scale reinforcement along the Waal River.

Note that the contextual scenarios are also applicable when mitigation strategy 2 is considered. Conversely, functional lifetime reductions of soil-based or construction-based dikes may also occur under mitigation strategy 1. To avoid redundancy, these combinations are not considered, and the analyses are presented separately.
Cost changes due to contextual scenarios can be directly added to the results of the functional lifetime analysis. Likewise, reductions in functional lifetime of inward and construction-based variants can be directly added to the LCC of the contextual sensitivity analysis. Only the outward variants with FLR acceptance require separate treatment, as changes in the FLR of affected dikes directly influence cost outcomes.

5.2. Results

5.2.1. Short-term (single instalment) cost comparison

Figure 5.6 presents the direct costs of the initial reinforcement and the subsequent reinforcement for each of the four conceptual dike alternatives, based on the assumptions and the reference scenario outlined in Section 5.1.1. Additionally, mitigation costs are included for the option of applying an asphalt layer to the crest of affected dikes. Accepting FLR cannot be illustrated within a short-term reinforcement context.

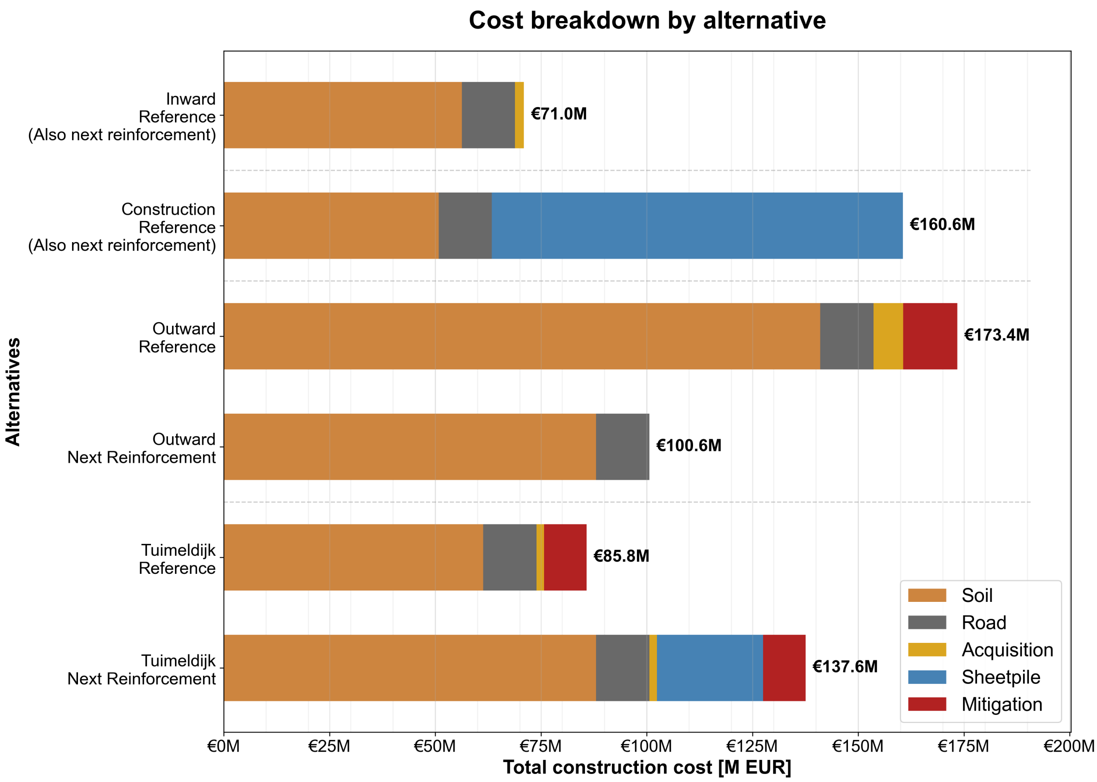

Figure 5.6: Cost of each dike alternative if reinforced for the whole Waal River. Also, the next reinforcement costs of the 20-metre outward expansion and Tuimeldijk are computed. All costs are direct costs and the costs are not converted to a net present value

The Tuimeldijk proves to be a promising alternative to construction-based reinforcement in terms of short-term costs, whereas the maximum 20-metre ODR is the most expensive option due to the large volume of soil required, although its cost remains comparable to conventional construction-based designs. In the reference scenario, the Tuimeldijk performs adequately and offers a relatively low-cost solution. However, its current design is only marginally sufficient, and in less favourable ground conditions, additional mitigation measures such as sheet piles may be required, increasing its cost. Although the 20-metre variant consists solely of soil, a relatively low-cost material, its extensive volume still makes it the most expensive option. This highlights that material choice alone does not determine cost-effectiveness, but the required quantity also plays a crucial role. Therefore, although this maximum expansion offers a high degree of adaptability, this may not always be desirable.
An optimisation between the maximum ODR (€173 million) and minimum Tuimeldijk (€86 million) presents a promising alternative to construction-based reinforcement (€161 million). Furthermore, as house demolition is not yet accounted for in this reference scenario, it may also present an interesting alternative to inward reinforcement if spatial constraints are considered. It should be noted that functional lifespan is not yet included in this comparison.

The next reinforcement costs, incurred after the initial instalment, show a significant cost reduction for the maximum 20-metre ODR. This is due to its adaptability: less soil is needed, no additional space acquisition is required, and no further mitigation measures are necessary. In contrast, the Tuimeldijk incurs higher direct costs for the next reinforcement, as increased hydraulic loads may lead to insufficient performance, necessitating additional sheet piles, space, and mitigation measures This suggests that the Tuimeldijk may defer these challenges to the future, potentially alleviating current time and budget constraints within the HWBP.

The mitigation costs associated with the flood defence system absorbing its own hydraulic impact are relatively minor, especially when compared to expensive alternatives such as floodplain creation. The cost of applying an asphalt layer to all affected dikes is approximately €15 million for the maximum 20-metre riverward expansion, which represents only a small fraction of the overall reinforcement costs. The mitigation costs for the Tuimeldijk are even lower, as its hydraulic impact is less severe. Moreover, planning and engineering costs for floodplain creation are not considered here, which further highlights the relative affordability and practical simplicity of the asphalt strategy.

5.2.2. Long-term cost-effectiveness comparison

This subsection compares the cumulative life cycle cost (LCC) of the four conceptual alternatives over a 100-year forecast horizon and evaluates the additional costs of two mitigation strategies under the reference scenario, as described in Subsection 5.1.1.

Strategy 1: Additional asphalt layer

Figure 5.7 presents the LCC of each alternative, associated with the reinforcement costs shown in Figure 5.6, and an asphalt layer is applied to all affected dikes as a mitigation strategy.

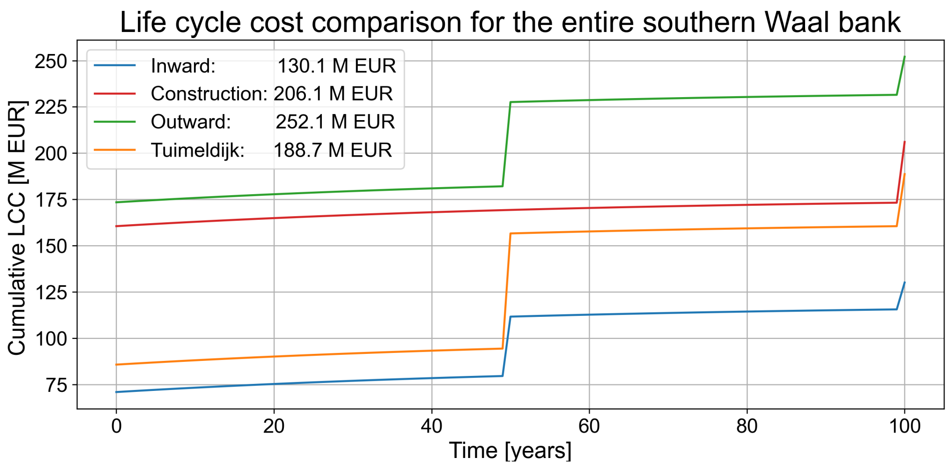

Figure 5.7: Cumulative LCC of the four alternatives for a fixed forecast horizon of 100 years under the reference scenario, and reinforcement of the whole southern Waal bank.

The minimum outward expansion variant, Tuimeldijk, proves to be a cost-effective alternative compared to the construction-based design over a 100-year lifetime, while the maximum 20-metre ODR variant is significantly more expensive. This range illustrates the potential of outward expansion as a more cost-effective yet adaptable and flexible alternative to construction-based reinforcement, depending on the degree of robustness and early investment integrated into the design. This balance between cost and adaptability reflects a trade-off between early expenses for long-term future uncertainties and current optimisation under HWBP budget constraints.

The design's functional lifetime has a significant impact on total cost and thus on its desirability. Although the next reinforcement of the construction-based alternative is costly, its long lifetime defers this expense far into the future, reducing its NPV impact. Conversely, if the functional lifetime of soil-based designs could be extended, their cost-effectiveness would improve. This appears more feasible and desirable for the 20-metre outward expansion than for the Tuimeldijk, as the former has demonstrated greater adaptability (see Section 2.3).

Strategy 2: Accepting FLR
As discussed in Subsection 5.1.3, the mitigation strategy that accepts functinoal lifetime reduction (FLR) is analysed for two different reinforcement lengths: one scenario considers reinforcement only for the reach Dreumel–Boven Leeuwen, while the other assumes reinforcement along the entire southern Waal bank, similar to the approach used in Strategy 1.

Reinforcement of the Dreumel–Boven Leeuwen section
Figure 5.8 presents the total LCC of the entire river system, including M&O and next reinforcement costs that could be affected by ODR, assuming the Dreumel–Boven Leeuwen (D–BL) section is the first instalment using one of the four alternatives. For outward expansion variants, FLR acceptance strategy is applied.

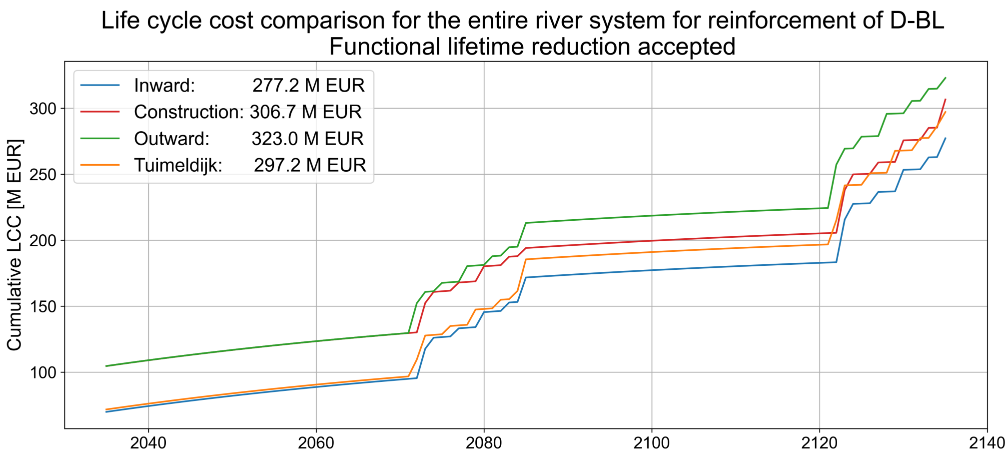

Figure 5.8: Cumulative LCC for the entire affected river system if the Dreumel–Boven Leeuwen reach is reinforced, per alternative.

Figure 5.8 illustrates how the outward variants trigger earlier reinforcement, resulting in a higher NPV due to advanced expenditures, and thus an increased LCC. Although the first reinforcements of dikes upstream of the D–BL reach are not simulated until 2073 (first end-of-lifetime of a dike segment), the timing shift remains relevant. If reinforcements occurred earlier, which is a feasible scenario for another river system where dikes are near the end of their functional lifetimes, cost differences would become more pronounced.

These additional costs, resulting from FLR acceptance, should be attributed to the riverward variants. Figure 5.9 breaks down the total system costs from Figure 5.8 into two components: the reinforcement costs per alternative for the D–BL section (left panel), alongside the additional LCC of upstream segments potentially affected by reduced functional lifespan (right panel). In the right panel, the total system cost of the inward variant, excluding reinforcement of the D–BL section, serves as a reference for the expected LCC of the river system. This reference is then compared to the implementation of ODR variants with FLR acceptance, and the cost difference is quantified. The total cost per alternative for the D–BL reach is the sum of both components.

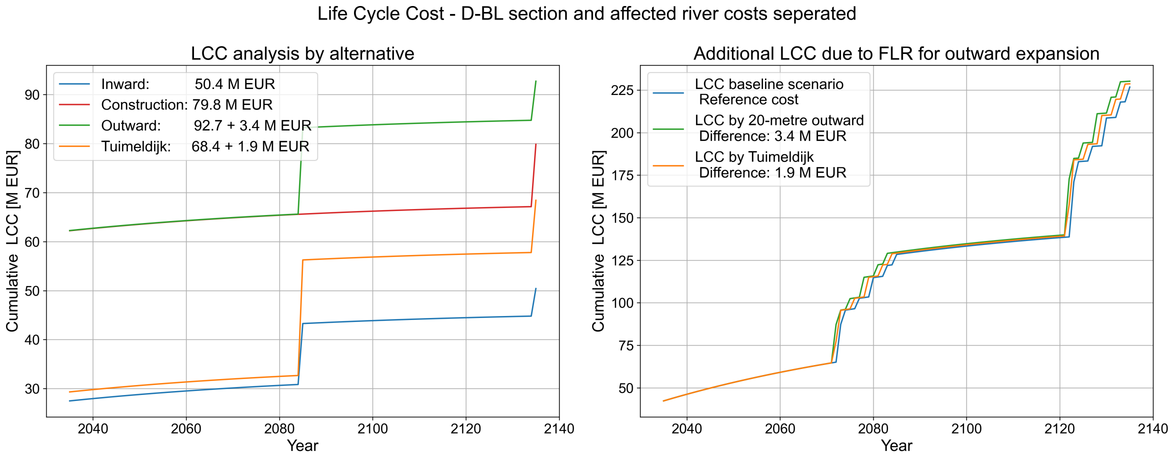

Figure 5.9: Cumulative LCC per alternative for the Dreumel–Boven Leeuwen reach. **Left:** costs per alternative for the considered reach only. **Right:** additional LCC of the affected dike segments resulting from acceptance of functional life reduction due to outward expansion.

Similar to the findings from Strategy 1, the Tuimeldijk is a cost-effective alternative compared to construction-based reinforcement, whilst the 20-metre ODR remains more expensive. Inward reinforcement without landward spatial constraints remains the most cost-effective option. Note that the differences in cost-effectiveness between the dikes are relatively smaller compared to the findings presented in Figure 5.7, while the reinforced section is shorter.

The additional mitigation costs to address the ODR-induced WLD by accepting FLR amount to only €3.4 to €1.9 million over the entire lifecycle, as illustrated in the right panel of Figure 5.9. The 20-metre ODR results in the highest mitigation costs through FLR acceptance, but these costs are relatively higher for the Tuimeldijk compared to the amount of WLD. This occurs due to the steep incline of the water level gradient at the start of the outward intervention, resulting in long adaptation lengths and thus great amount of dikes whose functional lifetime is reduced (see Subsections 3.2.4 and 3.2.5). Still, this is significantly lower than the €10 million cost of applying a mitigation asphalt layer, which makes outward expansion more cost-effective with this strategy, and particularly when compared to the potential cost required for floodplain mitigation.

It should be noted, however, that if upstream dikes are reinforced earlier, the impact of advancing expenditures would be greater. Therefore, the optimal strategy depends on the expected timing of upstream reinforcements. Furthermore, a large number of assumptions were made to compute the FLR, which may vary spatially. Still, this specific reference scenario of reinforcement at Dreumel–Boven Leeuwen is based on actual data, so these low mitigation costs can realistically occur.

Reinforcement of the entire southern Waal bank

As discussed, the same analysis is conducted assuming that the entire southern Waal riverbank is reinforced using the same alternative, to directly compare Strategy 2 to Strategy 1. This requires more assumptions of the upstream affected dike segments, as stated in Subsection 5.1.3.

Figure 5.10 presents the LCC of each alternative, including the additional LCC resulting from FLR acceptance for the outward expansion alternatives.

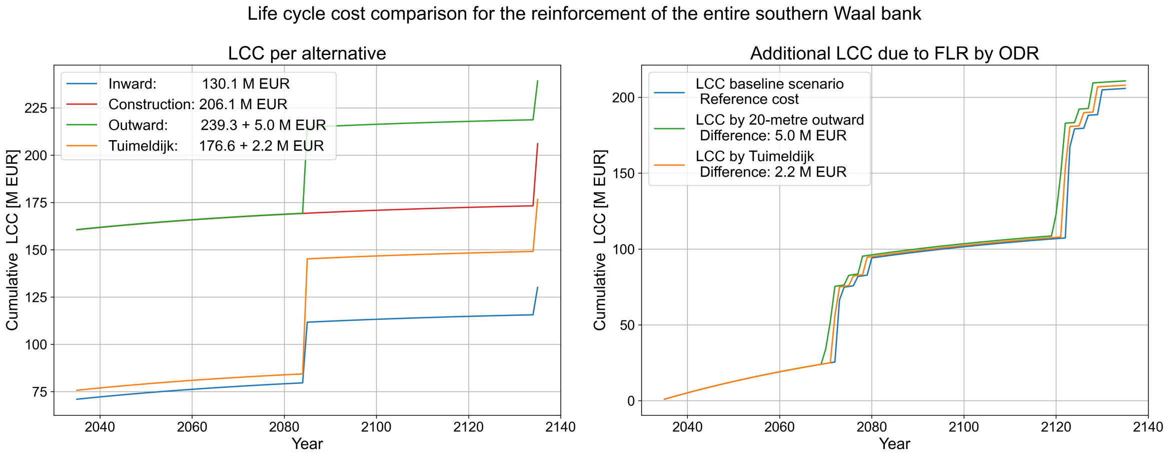

Figure 5.10: LCC of each alternative if applied to the whole southern Waal bank. **Left:** the LCC per alternative applied along the whole southern Waal bank. **Right:** The additional LCC of all potentially affected dikes upstream, with the inward variant as reference and the impact of FLR acceptance as mitigation strategy for outward expansion

Both outward expansion alternatives are several million euros less expensive compared to the strategy where the water level differences are mitigated using an asphalt layer, indicating that Strategy 2 is the more cost-effective strategy. The costs for the inward and construction-based alternatives are identical to those in Strategy 1, as expected, since these variants do not require mitigation of water level differences.

The additional life cycle cost of ODR has not increased significantly due to FLR acceptance, compared to the case where only the Dreumel–Boven Leeuwen reach is reinforced. This can be attributed to the adaptation length remaining largely unchanged, and the water level rise being less pronounced over longer intervention lengths. This is again explained by the steep gradient of water level differences for short interventions, and the rapid increase in adaptation length even for small water level changes. This demonstrates that longer implementation of ODR is relatively more cost-effective.

5.2.3. Sensitivity analysis

The reference scenario used in the previous LCC analyses relies on several assumptions, such as fixed and new soil prices, a fixed intervention length, the absence of spatial constraints, and specific assumptions regarding the functional lifetime. This section investigates alternative scenarios in which these assumptions are varied, as outlined in Subsection 5.1.4. An overview of the resulting LCC variations across all considered scenarios, strategies, and intervention lengths is provided in Figure 5.13, which is presented at the end of this analysis.

Recycled soil

Figure 5.11 shows the direct instalment and next reinforcement cost of each alternative under the reference scenario, with the assumption that recycled soil is applied instead of new material.

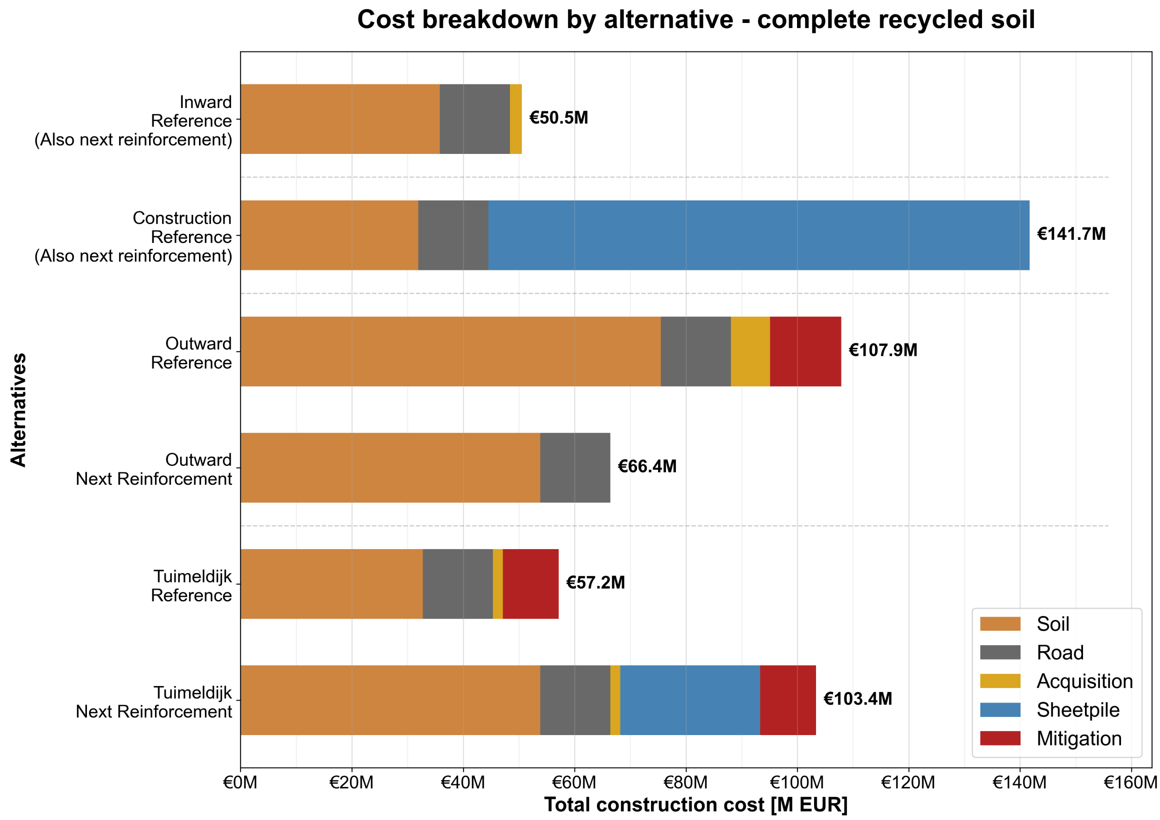

Figure 5.11: Cost of each dike alternative if it consists of complete recycled soil

Applying recycled soil significantly improves the cost-effectiveness of the maximum 20-metre outward variant, which is otherwise the most expensive option due to its high soil demand (as illustrated in Figure 5.6). With recycled soil, the 20-metre outward reinforcement becomes notably more cost-effective than the construction-based option, as shown in Figure 5.11. The Tuimeldijk also remains a promising alternative, although its next reinforcement costs remain high. If no spatial constraints are present, inward reinforcement continues to be the most cost-efficient option.

It is important to note that the projected cost savings depend heavily on the availability and quality of recycled soil. A substantial portion of soil-related costs arises from the use of high-quality clay, which is not always available in recycled material, particularly for longer intervention lengths. This limitation is less pronounced for recycled sand and potting soil [Van Heereveld, 2020], making the feasibility of full-scale application context-dependent.

Intervention length dependency
Figure 5.12 presents the direct instalment cost per alternative over a range of intervention lengths, where the total reinforcement costs are normalised per linear metre of dike.

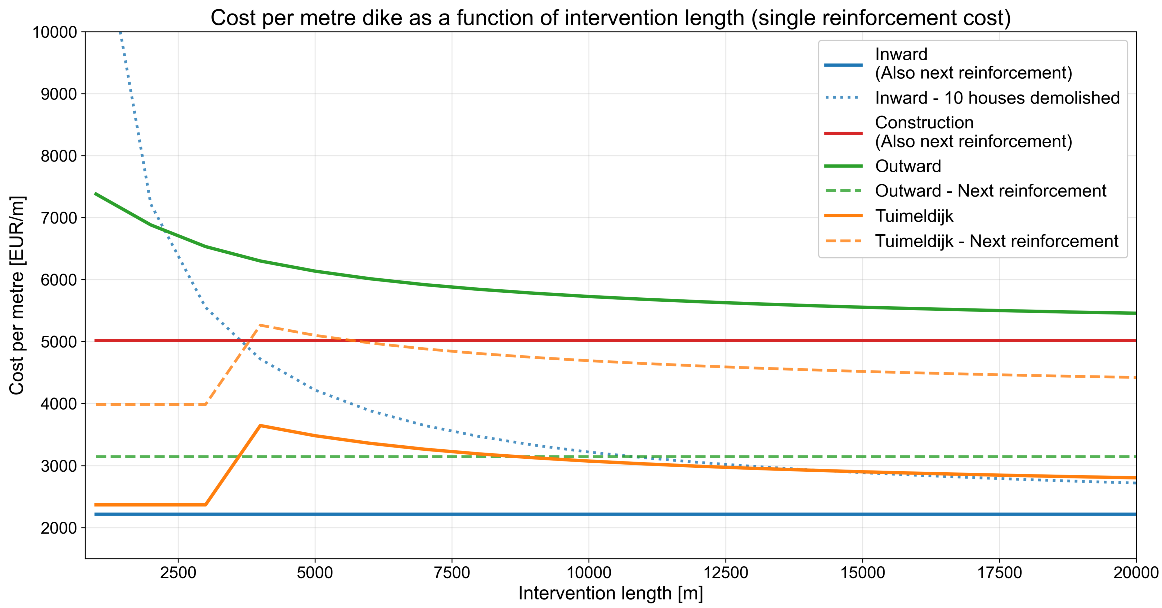

Figure 5.12: Cost per linear metre of dike for different reinforcement alternatives, shown as a function of intervention length. The costs are calculated by dividing the total reinforcement costs by the corresponding intervention length.

Outward dike reinforcement becomes more cost-efficient over longer stretches, as shown in Figure 5.12. Short interventions, however, are penalised by high mitigation costs concentrated over a limited reach. This sensitivity analysis confirms that these costs are driven by local hydraulic conditions previously discussed in Subsections 3.2.4 and 3.2.5, reinforcing the findings of Subsection 5.2.2.
Although longer implementation of ODR is generally more beneficial, the maximum 20-metre ODR variant never achieves lower direct instalment costs than construction-based reinforcement in the reference scenario.
For the Tuimeldijk, the intervention length must be sufficiently long for the direct costs of its next reinforcement to become more cost-effective than the construction-based variant. If the length is shorter than 3 kilometres, WLD remain below the 1-mm rule, eliminating the need for mitigation costs across the entire reinforced region. This is illustrated by the horizontal start of the orange lines. Applying the Tuimeldijk up to 3 km is therefore allowed under RBK regulations in this context, provided discharge capacity and nature values are not considered.

Spatial constraints, such as property acquisition for inward reinforcement, become disproportionately expensive when applied to short dike sections, as high fixed costs are distributed over fewer metres. This indicates that in extremely short, highly urbanised areas, ODR is more cost-effective than inward reinforcement. These effects underscore the importance of intervention length when evaluating the cost-effectiveness of ODR alternatives.

Scenario-based sensitivity
Table 5.4 presents the LCC at the end of a 100-year forecast horizon for the additional spatial and cost scenarios that influence reinforcement cost of the southern Waal bank per alternative, as discussed in Subsection 5.1.4. The reference scenario, as well as the analyses on recycled soil and intervention length presented earlier in this chapter, are incorporated.

Table 5.4: LCC comparison of the four conceptual dike designs across various spatial, cost, and intervention length scenarios, evaluated over a 100-year forecast horizon based on the application of Strategy 1. Green and red cells indicate scenarios that are 5 million euros more or less cost-effective, respectively, relative to the reference scenario within each design. The LCC of the designs under the reference scenario are colour-coded to reflect relative cost-effectiveness, ranging from green (most cost-effective) to red (least).

Scenario / Cost in M EUR	Inward reinforcement	Construction-based	20-metre ODR	Tuimeldijk (5-metre ODR)
If alternative applied to the whole Waal River				
Reference: Mitigation Strategy 1	130.1	206.1	252.1	188.7
Recycled soil	96.2	183.4	164.1	137.6
Piping sensitive soil	171.7	206.1	252.1	220.2
150 houses purchased inward	378.6	206.1	252.1	188.7
50 houses purchased inward	192.5	206.1	252.1	188.7
25% sheetpile cost increase	130.1	229.7	252.1	195.6
Future spatial pressure	344.7	206.1	252.1	220.9
If alternative applied to 15 kilometre reinforcement				
Reference: Mitigation Strategy 1	61.0	96.6	122.9	98.3
Recycled soil	45.1	86.0	81.6	74.4
Piping sensitive soil	80.5	96.6	122.9	111.0
30 houses purchased inward	110.7	96.6	122.9	98.3
Three scenario's combined	136.6	86.0	81.6	89.2
25% sheetpile cost increase	61.0	107.7	122.9	99.4
Future spatial pressure	185.4	96.6	122.9	111.3
If alternative applied for 5 kilometres reinforcement				
Reference: Mitigation Strategy 1	20.3	32.2	45.6	40.6
Recycled soil	15.0	28.7	31.9	32.6
Piping sensitive soil	26.8	32.2	45.6	44.8
10 houses purchased inward	36.9	32.2	45.6	40.6
Three scenario's combined	38.1	28.7	31.9	36.8
Future spatial pressure	61.8	32.2	45.6	44.9

An optimal design between the maximum 20-metre ODR variant and the minimum ODR Tuimeldijk, which does not require sheet piles, can be the most cost-effective solution in scenarios where the subsoil is prone to piping or spatial constraints exist on the landward side. It becomes particularly cost-effective when recycled soil is available or sheet pile prices increase. Notably, in the combined scenario involving recycled soil, piping-sensitive subsoil, and the demolition of ten houses, the maximum 20-metre ODR variant even slightly outperforms inward reinforcement. This highlights the potential of outward designs, particularly when multiple spatial, material, and soil constraints coincide.

ODR over long intervention lengths is cost-effective compared to construction-based alternatives, with a tipping point around 15 kilometres. For reinforcement along the entire Waal River, an optimised outward variant positioned between the two extremes may offer a viable soil-based alternative to construction-based solutions. Using the Tuimeldijk as the starting point, the optimisation range in the reference scenario lies between approximately €188.7 million and €206.1 million.
The 20-metre ODR is robust and relatively insensitive to scenario variations, but it remains the most expensive option if recycled soil cannot be used, due to the substantial volume of soil required for installation and the associated high NPV. The Tuimeldijk performs well in several scenarios but is more vulnerable to piping and future spatial pressure. A well-balanced outward design that mitigates piping without requiring sheet piles, incorporates adaptability, but avoids excessive robustness and soil volumes where not needed, could offer a resilient and cost-effective solution.

The implementation of the Tuimeldijk, and thus the general application of ODR, becomes more cost-effective than inward reinforcement when demolition of houses on the landward side is required, if this is even considerable. Excluding demolition in densely built areas, the cost of inward reinforcement already rises to €192.5 million. When spatial pressure is taken into account, these costs increase significantly over time, making ODR clearly the more cost-effective soil-based alternative. This underscores the need for a viable soil-based dike alternative, as inward reinforcement may become increasingly untenable in future scenarios.

At 15-kilometre intervention lengths, typical for single dike projects, the cost-effectiveness of ODR shifts. In the reference scenario, both ODR variants become less attractive than construction-based reinforcement. However, under cumulative pressures and with the availability of recycled soil, the soil-based maximum 20-metre ODR variant becomes competitive, or even superior, to all other alternatives. This represents a critical transition point: from this intervention length onwards, outward designs can outperform non-outward solutions, provided they are optimised to avoid future sheet pile requirements and non-outward variants experience spatial or material constraints.

For short intervention lengths, illustrated here for a 5-kilometre reach, ODR variants are consistently penalised due to steep water level gradients, as previously concluded. Except when future spatial pressure is considered, inward reinforcement remains more cost-effective, even under unfavourable scenarios. The Tuimeldijk, while initially appearing attractive, becomes less viable due to its limited adaptability and the likely need for future sheet piles. As such, ODR is generally not a desirable option for short reinforcement lengths and is less cost-effective than construction-based alternatives, unless non-monetary or long-term planning flexibility considerations are prioritised.

Functional lifetime related sensitivity

Table 5.5 presents the LCC outcomes under various functional lifetimes and scenarios related to functional lifetime reduction (FLR), as introduced in Subsection 5.1.4. These results are based on full-length reinforcement and reinforcement of the Dreumel–Boven Leeuwen section, and do not further account for variations in intervention length or the contextual scenarios discussed previously. Nevertheless, it should be noted that these contextual scenarios are also applicable when Mitigation Strategy 2 is selected. Conversely, cost implications resulting from changes in functional lifetime are also relevant for Strategy 1, as elaborated at the end of Subsection 5.1.4.

Table 5.5: LCC comparison of the four conceptual dike designs under varying scenarios affecting the FLR, based on Strategy 2. Green and red cells indicate scenarios that are 5 million euros more or less cost-effective, respectively, relative to the reference scenario within each design. The LCC of the designs under the reference scenario are colour-coded to reflect relative cost-effectiveness, ranging from green (most cost-effective) to red (least).

Scenario / Cost in M EUR	Inward reinforcement	Construction-based	Outward 20-metre	Tuimeldijk
If alternative applied to the whole Waal River				
Reference: Mitigation Strategy 2	130.1	206.1	244.3	178.8
Annual subsidence reduction 1 mm/year	130.1	206.1	246.0	179.5
Annual subsidence increase 1 cm/year	130.1	206.1	243.1	178.8
Low-end climate change scenario	130.1	206.1	247.0	179.5
Severe climate change scenario	130.1	206.1	244.1	178.8
Functional lifetime soil +20 years	107.1	206.1	209.9	134.1
Functional lifetime construction -20 years	130.1	218.3	245.1	179.1
Combined with +25% sheetpile cost	130.1	243.5	245.4	181.7
Functional lifetime construction -50 years	130.1	245.8	246.8	179.9
Functional lifetime all variants -20 years	147.5	218.3	270.8	212.5

Scenario / Cost in M EUR	Inward reinforcement	Construction-based	Outward 20-metre	Tuimeldijk
All affected dikes soil-based	130.1	206.1	243.3	178.4
All affected dikes construction-based	130.1	206.1	245.1	179.1
Combined scenario: worst case FLR	130.1	206.1	250.1	180.2
Combined scenario: climate pressure	130.1	218.3	245.9	179.5
If alternative applied to 12.4 kilometre reinforcement				
Reference: Mitigation Strategy 2	50.4	79.8	95.1	70.3
Complete 20-year FLR	57.1	84.6	107.0	83.8
All affected dikes soil-based	50.4	79.8	96.7	70.6
All affected dikes construction-based	50.4	79.8	95.5	70.0

The timing of the next reinforcement of affected dike segments is the most decisive factor for the cost-effectiveness of ODR with FLR acceptance as a mitigation strategy, particularly when reinforcements are scheduled close to the present. In such cases, the benefits of discounting on the forward expenditures resulting from FLR acceptance are limited, making ODR less attractive. Conversely, when reinforcements are planned far into the future (e.g. more than 30 years), ODR becomes more cost-effective. This pattern holds across dike types, as the trade-off between higher costs and longer lifespans of construction-based alternatives versus lower costs and shorter lifespans of soil-based alternatives tends to balance out.

Mitigation Strategy 2, even with uncertainty in variables that govern FLR, is the more cost-effective strategy, while ODR variants are robust to uncertainties in FLR and dike type characteristics, provided that next reinforcements occur far in the future. In the reference scenario, combined with variations in subsidence and climate scenario (i.e. annual HBN slope), the sensitivity of ODR's cost-effectiveness to changes in functional lifetime is limited. Even in a scenario where all affected segments have a 20-year FLR and minimal subsidence, accepting this FLR results in only €3 million additional cost compared to the original functional lifetime (scenario not shown in the table). This confirms that under Mitigation Strategy 2, the cost-effectiveness of ODR is not significantly influenced by FLR duration or the type of dike present at the affected reaches.

Furthermore, under Strategy 2, ODR variants remain more cost-effective even for shorter intervention lengths. While Strategy 1 results in the Tuimeldijk becoming less cost-effective than the construction-based variant at an intervention length of 15 km, Strategy 2 maintains its cost-effectiveness well below this threshold. Since the Tuimeldijk still remains €9 million more cost-effective at 12.4 km, it can be reasonably assumed that ODR becomes the more cost-effective option at intervention lengths starting from approximately 10 km. This is because, for short intervention lengths, the resulting WLD over the adaptation length does not lead to significant FLR acceptance costs. However, under Strategy 1, an asphalt layer is required over the entire adaptation length, which turns out to be considerably more expensive than accepting the FLR for shorter interventions.

Outward variants are better suited to accommodate plausible changes in functional lifetime than construction-based alternatives. If soil-based dikes can be extended beyond 50 years, or if construction-based dikes fall short of their expected 100-year lifespan, the desirability of ODR increases. These scenarios are more realistic than the reverse, reinforcing the resilience of outward designs to future climate-related uncertainties. Moreover, even in the worst-case scenario where all dike alternatives experience a reduction in functional lifetime, the Tuimeldijk variant remains the more cost-effective option.

Figure 5.13 presents a consolidated overview of the cost-effectiveness of all dike variants across the considered scenarios, and highlights the range of cumulative LCC values per alternative and intervention length. The markers directly represent the LCC values listed in Tables 5.4 and 5.5, and serve to visualise previously discussed results.

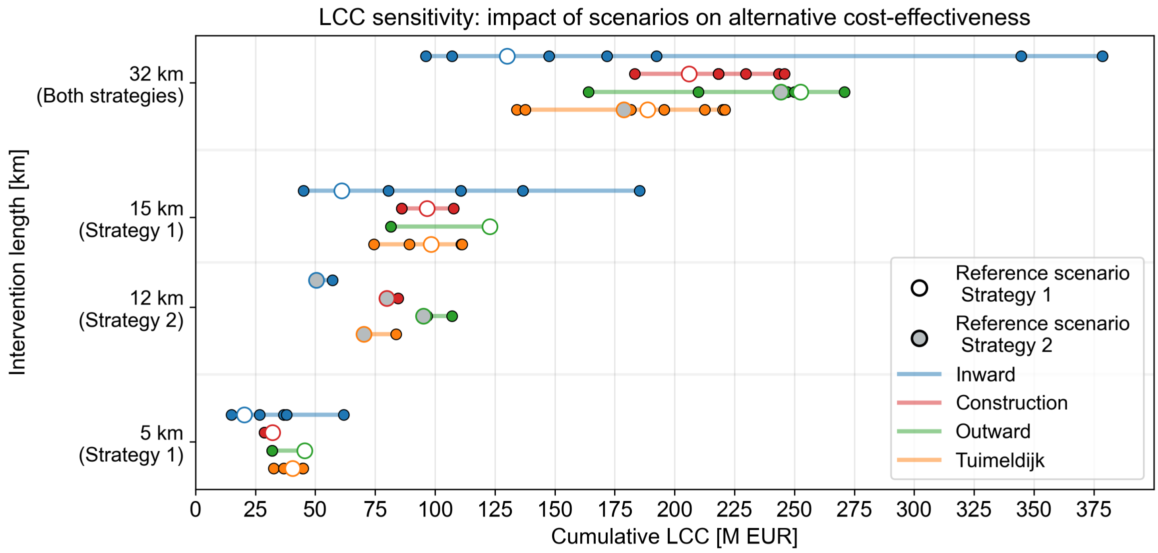

Figure 5.13: Overview of cumulative LCC range for all dike reinforcement variants across multiple scenarios and intervention lengths, based on the sensitivity analysis. The black and grey markers show the LCC for the reference scenarios, with either Strategy 1 or Strategy 2, respectively.

5.3. Main insight on the cost-effectiveness of ODR

An optimal outward design positioned between the two extremes offers the most cost-effective solution: a design close to the Tuimeldijk variant but without requiring sheet piles, preferably also not in future reinforcement. This design becomes particularly attractive in scenarios involving landward spatial pressure, piping-sensitive subsoil, rising steel prices, or uncertainties regarding the actual functional lifetime of dike types.

If sufficient recycled soil is available, outward designs become more cost-effective than construction-based alternatives and can, under certain scenarios, directly compete with inward reinforcement. Fully soil-based ODR designs, if not exceptionally but sufficiently robust, offer a flexible and adaptable alternative. Although not consistently cheaper, they approach the cost-effectiveness of construction-based solutions and offer advantages in recyclability and long-term adaptability.
Furthermore, while not empirically demonstrated and based on the conditions of the case study (Appendix C), the results suggest that ODR, when absorbing its own hydraulic impact, may be more cost-effective than ODR combined with floodplain mitigation.

Accepting FLR proves to be the most cost-effective strategy, provided that the next reinforcement of affected dikes lies sufficiently far in the future. In such cases, uncertainties in FLR duration and the type of dike present at the affected reaches have limited impact on cost-effectiveness.

Longer intervention lengths, starting from approximately 10 kilometres, significantly improve the cost-effectiveness of ODR. In contrast, short stretches are penalised due to high WLD per unit length and associated mitigation costs. Mitigation Strategy 2 is already more cost-effective than construction-based reinforcement for shorter intervention lengths compared to Strategy 1, which only becomes cost-effective after 15 kilometres. This is because even small WLD result in long adaptation lengths, requiring extensive asphalt layers under Strategy 1, which are considerably more expensive than accepting FLR under Strategy 2.

In short-term scenarios, ODR is only cost-effective if the design closely resembles the Tuimeldijk variant or incorporates recycled soil. Otherwise, high soil costs make it less competitive than non-outward alternatives. However, over a longer forecast horizon, outward variants are better suited to accommodate plausible changes in functional lifetime than construction-based alternatives. If soil-based dikes can be extended beyond 50 years, or if construction-based dikes fall short of their expected 100-year lifespan, the desirability of ODR increases. These scenarios are more realistic than the reverse, reinforcing the resilience of outward designs to future climate-related uncertainties.

6
Discussion

This study set out to examine whether outward dike reinforcement (ODR) can serve as a technically and economically viable alternative to conventional reinforcement strategies. Focusing on the Waal River and employing four conceptual dike designs, the research quantified the hydraulic impacts of ODR, evaluated its influence on ODR feasibility based on relevant failure mechanisms of affected dikes, and translated these effects into a comparative cost-effectiveness analysis against non-outward variants. The objective was to determine whether ODR, when the flood defences accommodate the hydraulic consequences, could be both technically feasible and cost-effective, thereby broadening the current range of reinforcement options beyond those permitted under the Rivierkundig Beoordelingskader (RBK). The main insights and synthesis are presented at the end of the three analytical chapters. This chapter discusses the academic relevance of the findings, the broader societal and implementation implications of applying ODR beyond the current RBK regulatory framework, and the study's limitations.

6.1. Scientific contribution and implementation implications

Academically, this research makes a significant contribution by providing one of the first broad quantitative analyses of the hydraulic impact of riverward reinforcement in the Netherlands, beyond the scope of local river effects of reinforcement projects. It identifies governing factors such as floodplain width, roughness, intervention length, and location dependency. Furthermore, this study provides a first-order approximation tool that engineers can use in early design stages. Although applied to the Waal, the methodology can be transferred to other Dutch rivers once calibrated against reference D-Hydro runs, thereby supporting broader reinforcement planning. The results, therefore, expand both the scientific knowledge base and the design space available to practitioners.

ODR is particularly relevant in situations where conventional strategies encounter social resistance or face severe spatial constraints. In such contexts, costs are already expected to be significantly elevated, making the adaptive nature of ODR a potentially more attractive "low-regret" option. Moreover, ODR is currently often disregarded as an alternative due to the complex and costly mitigation measures required to comply with the 1-mm water level difference (WLD) rule stipulated by the RBK.
If ODR is applied where the flood defences absorb the hydraulic impact, it may become a more favourable alternative. This applies while under many scenarios, if applied for a considerably long intervention length, an optimised ODR variant that is sufficiently robust and does not require sheet piles, is more cost-effective than construction-based alternatives. Additionally, unlike sheet pile solutions, soil-based ODR retains flexibility for future modifications, which is increasingly important under conditions of climate uncertainty, as suggested by the Kennisprogramma Zeespiegelstijging.

However, applying this ODR alternative as a standard reinforcement strategy along extended river sections would reduce floodplain conveyance and storage capacity, which is currently not permitted under the RBK unless adequately mitigated. Furthermore, ODR offers limited opportunities for co-benefits such as nature development or water quality improvement, which have been central to the Room for the River programme and which also contributes significantly to climate resilience.

Therefore, ODR without floodplain mitigation should not be considered a default or first-choice solution. Its use must remain selective and context-driven, particularly in areas where conventional strategies are infeasible due to spatial or societal constraints and become disproportionately expensive. In such cases, the flexibility of ODR makes it a suitable complementary solution. It may help to alleviate the budgetary and time constraints currently experienced within the Flood Protection Programme. Furthermore, its adaptability allows for future modifications, should additional Room for the River measures or similar strategies become necessary, thereby preserving opportunities for climate-resilient development.

6.2. Limitations

Every study relies on assumptions and simplifications. The results provide meaningful insights into the relative effects and potential of outward dike reinforcement, but several methodological and scope-related limitations should be acknowledged.

Simplified 1D compound channel model

The hydraulic impact assessment is based on a simplified 1D compound channel model calibrated against D-Hydro simulations. The 1D model reliably reproduced general and averaged trends consistent with the D-Hydro outputs, although it consistently produces slightly conservative results due to its formulation.

Although the 1D model was able to simulate the maximum feasible Rhine discharge of 18,000 m^3/s, calibration was only feasible up to 17,000 m^3/s and could not be independently validated in most cases. For great amounts of outward expansion (e.g. +15 metre), the 1D model computes a more conservative value of WLD at the 17,000 m^3/s discharge compared to the D-Hydro simulations. At the maximum discharge, this conservative assumption is likely to be even stronger. This suggests that the design graphs may be overly conservative. While this is not necessarily problematic and did not limit the feasibility assessment of ODR, it does indicate minor mismatches that could be critical in relation to the 1-mm rule.

Furthermore, there is a slight mismatch in the used length when the entire Waal range is analysed. In the 1D model and D-Hydro simulations, the full length is modelled as 34 km, which is the correct length of the considered reach. However, in some subsequent analyses in the following chapters, an outdated value of 32 km is applied, as not all analyses could be redone after the error was discovered. This makes the findings from the 1D model appear extra conservative.

Although the water level differences are consistent with expectations and the D-Hydro outputs, and the calibrated variables are realistic for the Waal, calibration alone does not constitute full validation. This limitation must be acknowledged. Nevertheless, the model's computational efficiency and the absence of complex schematisation adjustments make it a valuable tool for preliminary design.

Lastly, the averaged findings from the 1D model were computed only for the Waal River and were not generalised to assess the relative impact across other Dutch river systems. An initial attempt was made to generalise the results by plotting them against the percentage of floodplain width claimed by ODR, but this did not yield consistent WLD outcomes across different expansion magnitudes and was therefore deemed unsuitable. A more promising approach could be to use the percentage of relative lost conveyance capacity. However, since the 1D model was calibrated specifically for the Waal River and due to time constraints, the findings were kept specific to this river to ensure clarity and reliability.

Adaptation length definition

The estimation of the adaptation length and its endpoint definition proved highly sensitive to methodological choices. Results depended not only on the definition of the endpoint and the strictness with which thresholds were applied, but also on the exponential form assumed through the empirical approximation of Bresse's analytical model. This means that small water level differences produce shallow tails and disproportionately long adaptation lengths, while larger differences yield steeper tails and shorter lengths. Consequently, short intervention lengths are strongly penalised with high WLD per unite length due to the backwater effects. This has a quite significant impact on the cost-effectiveness of ODR on short intervention lengths.

In addition, the spatial range of the D-Hydro simulations was insufficient to capture the actual end of the adaptation process. Approximation therefore relied on a heavily fitted approach, which produced significantly conservative values. Furthermore, the defined threshold applied was 0.1 mm, significantly stricter than the 1-mm rule of the RBK, and may thus be overly conservative. This bias was reflected in the adaptation length design graph (Figure 3.17), which likely overestimates adaptation lengths.

This has a direct impact on the implications of ODR, as such long adaptation lengths currently always reach the Pannerdensche Kop bifurcation and may therefore alter discharge distribution, which under RBK regulations is only permitted to deviate by 20 m^3/s [Van Doorn and Rijkswaterstaat Water, Verkeer en Leefomgeving, 2023]. These limitations underline the importance of establishing a clear and consistent definition and formulation for adaptation length, including the strictness of its endpoint, before ODR can be consistently assessed in practice or policy.

D-Hydro constraints and excluded criteria

The D-Hydro schematisations display several instabilities. The first arose near WL_872, where a grid fault produced a clear pulse in all simulations. Although the water level differences up to this point appear logical and consistent with expectations, this artefact indicates that parts of the schematisation are affected by errors and should be interpreted with caution.

A second issue resulted from the conversion of Baseline 7 into the D-Hydro environment. While the underlying cell values remained unchanged, altered indexing prevented additional analysis of cross-flows and discharge differences, both of which are formal RBK criteria. Consequently, the effect of outward expansion on discharge distribution at Pannerdensche Kop could not be quantified. This limitation is critical, as peak flows at this bifurcation have been proposed to trigger tipping points in discharge distribution, as hypothesised following the 1993 and 1995 floods [Blom et al., 2024]. Such dynamics could strongly influence the feasibility of ODR in the Waal.

Finally, morphological effects are excluded, as their assessment requires specialised modelling frameworks such as Delft3D, which are beyond the scope of this study. Deterministic calculations would be impractical given the strong influence of local variability. These omissions limit the completeness of the current assessment of ODR under the RBK framework.

Feasibility assessment with conceptual dike designs

The feasibility of ODR, based on the failure mechanisms of affected dikes, is primarily investigated using conceptual dike designs. These mechanisms are sensitive to a wide range of variables, whose influence is explored through conservative assumptions and broad parameter ranges. However, the designs remain hypothetical and context-specific. It cannot be ruled out that, outside the investigated range, a specific and detailed real-world scenario could lead to different outcomes. This uncertainty is partly due to the lack of detailed information on individual dike sections along the Waal, and a full-scale validation across all dikes is beyond the scope of this thesis.

For overtopping, the analysis show that under a wide range of conservative assumptions and design implications, the hydraulic impact of ODR on this mechanism is negligible. While this supports the feasibility of ODR in general terms, it cannot be guaranteed that overtopping is irrelevant under all possible local conditions. A unlikely but specific combination of factors could still result in overtopping, although such cases are not identified within the scope of this study.

For backward internal erosion, the expected benefits of ODR are less apparent due to simplified seepage paths, conservative assumptions, and anonymised soil data. A brief sensitivity analysis revealed that the assumption of uniform seepage length, without distinguishing between foreshore, and hinterland components, significantly influenced the results. In particular, the 20-metre ODR variant may in reality be more resistant to piping than simulated. If this improved resistance were accounted for, it could allow for a more severe piping scenario in which the 20-metre variant still performs adequately without requiring sheet piles. In contrast, inward and Tuimeldijk variants would likely require longer sheet pile lengths under such conditions. This would increase the relative benefit of the additional berm in the 20-metre ODR design and strengthen its desirability.

Furthermore, soil characteristics likely play a more decisive role in piping vulnerability than the minor hydraulic differences identified in this thesis. This is not clearly reflected in the current results, as the use of conceptual designs reduces the visible differences in piping sensitivity between alternatives. Measures such as floodplain lowering for water storage potentially have more severe consequences for piping risk than the millimetre-scale water level increases caused by ODR.

Stability is assessed qualitatively through expert judgement rather than geotechnical modelling, and only under stationary conditions that do not capture changes in soil strength. These simplifications reduce the precision of the failure mechanism analysis and may lead to an overestimation of stability for certain alternatives. A more detailed geotechnical assessment could therefore alter the outcomes and, by extension, influence cost comparisons. However, such in-depth stability calculations are beyond the scope of this hydraulic study.

Functional lifetime reduction

Functional lifetime reduction (FLR) is estimated under linear assumptions for crest subsidence and HBN increase. In reality, settlement is non-linear, typically higher shortly after construction and lower towards the end of the design life, which may result in the reduction in functional lifetime being more severe than currently estimated.
Moreover, only two future HBN reference points (2050 and 2100) are available, which prevents robust trend fitting, while climate change could introduce non-linear trajectories that further affect FLR. The heave limit state is also likely influenced by such non-linearity, making the dike more prone to lifetime reduction at the end of its design life. In the current analysis, it follows the linear HBN trend.

Additionally, adding an asphalt layer (Strategy 1) to extend the functional lifetime of affected dikes could also be applied to dikes not impacted by ODR, provided that stability and piping mechanisms do not lead to failure. In that case, Strategy 1 essentially becomes a matter of functional lifetime difference, where all dikes receive an asphalt layer. The main distinction then lies in the timing and cost of future reinforcements, as well as the total possible surplus height per dike segment. These aspects are more sensitive to assumptions and simplifications than the direct costs of adding asphalt.
Therefore, this approach is not considered, and the strategy is based on retaining the originally assumed functional lifetime, allowing the mitigation costs of this practical strategy to be benchmarked. However, it must be noted that this approach implicitly constitutes a functional lifetime reduction (FLR) assessment and could therefore result in different mitigation costs for this strategy.

For heave, the FLR analysis assumes that the limit state at the end of the design life is sufficient, represented by an exact sheet pile length. This approach was also applied to inward alternatives. While somewhat artificial, as it results from conceptual designs and projected reinforcement requirements, it does not substantially affect the overall findings.

Reinforcement costs are projected far in the future, which reduces their weight in the LCC assessment. For reinforcements closer to the present, however, these uncertainties would be more significant. Such near-term cases could not be analysed due to limitations in HBN data and the setup of the FLR code, which also restricts the sensitivity of the mitigation analysis.

Cost-effectiveness analyses

The cost-effectiveness analysis is subject to several methodological limitations. First, all uncertainties in the hydraulic assessment, adaptation length, and FLR translate directly into the cost functions. In particular the adaptation length has a significant influence on the cost-effectiveness of ODR, especially when an asphalt layer is selected as the mitigation strategy.

Additionally, the conceptual dike designs were developed using conservative assumptions to capture the maximum hydraulic impact and assess feasibility. While this approach supports robustness in the analysis, it also affects cost outcomes. For instance, the 20-metre ODR variant may have been designed more robustly than necessary, potentially leading to an overestimation of its costs, even for the maximum ODR variant.

Moreover, the simplified assumptions regarding seepage length, particularly the lack of distinction between total, foreshore, and hinterland components, may underestimate the actual resilience of the 20-metre variant. While other variants would require significant sheet piles under these conditions, the maximum 20-metre variant would not. This would potentially increase the relative benefit of the additional berm in the 20-metre ODR design and improve its cost-effectiveness compared to the other variants in the piping scenario.
Nevertheless, to maintain consistency across the analysis, design choices were kept fixed for the cost-effectiveness study. While this was necessary for comparability, it may have led to an under-representation of the potential advantages of soil-based ODR under more extreme conditions.

The cost-effectiveness analysis considers only direct construction costs. Variability in operation and maintenance costs was excluded due to limited data availability, even though such differences could meaningfully affect the comparison between alternatives.
Environmental and life-cycle-related costs, such as CO_2 emissions, transportation, and material production, were also not included, despite their potential significance. Engineering, project management, and risk-related costs were similarly omitted. This is particularly relevant, as ODR may, in practice, reduce project complexity and thereby lower delays and indirect costs, an effect that cannot be quantified within the current framework.

Furthermore, in this study, no cost comparison could be made between cases where mitigation involves strengthening affected dike sections and those requiring floodplain measures, as no data was found regarding the costs of these floodplain mitigation efforts. This comparison, especially if it would include engineering and planning costs, could prove highly beneficial for the desirability of the investigated ODR variant of this thesis. The potential cost differences between these approaches were not quantified but may substantially affect the comparative outcomes.

Finally, scenario analyses for factors such as land acquisition, housing, or material costs are purely hypothetical. These serve to illustrate potential sensitivities but cannot be supported by empirical sources, and should therefore be interpreted with caution.

7 Conclusions

This chapter presents the conclusions of the thesis. The research investigated whether outward dike reinforcement (ODR) can be considered a technically feasible and cost-effective alternative under a relaxed regulatory framework, where the flood defence system compensates for ODR-induced hydraulic impact without strictly applying the one-millimetre water level limit from the Rivierkundig Beoordelingskader (RBK). The study is conducted in the context of the reinforcement of the southern Waal bank, using four self-developed conceptual dike designs to explore the effects of ODR. The investigation was structured around three subquestions, addressed in a logical sequence, which together support the answer to the main research question.

Subquestion 1: What is the hydraulic impact of outward dike reinforcement, and which factors most significantly influence the resulting water level differences?

The ODR-induced water level difference (WLD) in the Waal ranges from 0 to 4 centimetres, depending on the implementation and floodplain characteristics, based on the simplified 1D compound channel model.
D-Hydro simulations confirm these trends, showing a maximum increase of 2.3 centimetres for the Waal under the maximum 20 metre outward expansion. The relationship between outward expansion and WLD is observed to be practically linear.

The most influential factors in determining WLD are floodplain width and roughness, as well as the location and length of the riverward intervention. Narrow floodplains experience the greatest relative loss of conveyance capacity, resulting in disproportionately higher WLD. For floodplains narrower than 2 kilometres, smooth surfaces lead to higher ODR-induced WLD than rough ones.
Short ODR implementations are penalised by high WLD per unit of intervention length, caused by a sharp local water level gradient at the start of the intervention. Still, longer continuous reinforcements result in the most considerable overall water level increases, as the greater length allows the water to approach its new equilibrium depth more closely compared to shorter or discrete interventions. Location also plays a decisive role, with expansions near hydraulic bottlenecks generating the most pronounced relative effects.

Adaptation lengths extend over several tens of kilometres, even for modest increases in water level, and are influenced by the amount of WLD. Their exact extent, however, depends on how strictly the endpoint threshold is defined.

Subquestion 2: To what extent is outward dike reinforcement technically feasible, considering its impact on the performance and functional lifetime of affected dikes, and how can these impacts be mitigated?

Outward dike reinforcement is technically feasible, given the performance of affected dikes, as the hydraulic impact does not lead to immediate failure of any of the main failure mechanisms. Of these mechanisms, only the overflow and heave sub-mechanisms fail to meet safety requirements at the end of the intended functional lifetime, resulting in functional lifetime reduction (FLR). The hydraulic impact on overtopping is negligible, and overall stability is not compromised as long as the WLD remains below 10 centimetres, a threshold not approached under the maximum 20-metre expansion variant. The heave mechanism is only affected if dikes are designed with minimal robustness; otherwise, the hydraulic impact has no effect on this mechanism either.

The extent of FLR is governed by the annual increase in hydraulic load level (HBN) and the magnitude of the WLD, with low annual HBN increases and high WLD resulting in greater FLR. For overflow, the rate of dike subsidence also affects FLR, with low subsidence resulting in significantly higher FLR. For the heave criterion, additional robustness is decisive. If the depth of the blanket layer or sheet piles exceeds the required value by the WLD or half the WLD, respectively, no FLR due to heave will occur. Future reinforcements can be designed accordingly to prevent further lifetime loss.

For the Waal, the maximum 20-metre outward expansion results in a lifetime reduction of 4.3 years due to heave, and 2.2 years due to overflow. Considering a modest degree of robustness in the dikes, heave is not governing, making a reduction of approximately 2.2 years the most representative.

Two strategies are proposed to mitigate the effects of the hydraulic impact. The first is to apply an additional asphalt layer, which is practical, can be implemented during planned maintenance, and symbolises the additional safety. The second is to accept the functional lifetime reduction as part of long-term planning, but requires detailed data on the affected dikes. Together, these strategies ensure that functional lifetime reduction does not obstruct the feasibility of outward reinforcement.

Subquestion 3: How does outward dike reinforcement compare to non-outward alternatives in cost-effectiveness, both within a single reinforcement cycle and over a longer forecast horizon?

For a single dike reinforcement cycle, riverward expansion is a cost-effective alternative compared to a construction-based variant if an optimal design closely resembles the Tuimeldijk variant without sheet piles, or if recycled soil can be used. Otherwise, excessive soil volumes and associated costs render outward expansion too expensive. However, if designed with adaptability in mind, future reinforcement costs can be significantly lower, as space and soil have already been reserved.

Over a 100-year forecast horizon, an optimised design of outward dike reinforcement offers a cost-effective alternative to construction-based reinforcement. Positioned between the minimum (5-metre) riverward-expanded Tuimeldijk and the maximum 20-metre ODR variant, this design should not require sheet piles, allows space for future adaptability, and is suitable for intervention lengths exceeding 10 kilometres.
The maximum 20-metre variant is not prone to piping or future uncertainties, but comes at the expense of higher upfront costs, making such a robust design less desirable. The Tuimeldijk variant is cost-effective if it does not require sheet piles, but is more vulnerable to future uncertainties and lacks flexibility in its design. Still, the higher cost of its next reinforcement has a smaller impact on overall cost-effectiveness, as these costs are discounted due to their occurrence far in the future. This may be desirable under the current budgetary and time constraints of the Flood Protection Programme.

The cost-effectiveness of an optimised ODR increases in scenarios where landward space behind the dike is constrained, the subsoil is prone to piping, or where uncertainties exist in material properties or functional lifetime, especially if recycled soil can be used. In such cases, ODR proves to be more cost-effective than construction-based reinforcement or inward reinforcement.

Accepting FLR as a mitigation strategy is the more cost-effective of the two, making an optimised ODR variant more cost-effective than construction-based reinforcement for intervention lengths starting from approximately 10 kilometres. The cost-effectiveness of ODR under this strategy is not significantly affected by the sensitivities that govern FLR or by misidentification of the affected dike type. However, to preserve this cost-effectiveness, the next reinforcement of affected dikes must be scheduled sufficiently far into the future, as otherwise the benefits of discounting on the forward expenditures resulting from FLR acceptance are limited. If an additional asphalt layer is applied instead, ODR only becomes cost-effective for intervention lengths exceeding 15 kilometres.

Main research question: To what extent is outward dike reinforcement under a relaxed regulatory framework of the 'Rivierkundig Beoordelingskader' a feasible and cost-effective primary flood defence?

Outward dike reinforcement, with the flood defence system absorbing the hydraulic impact, proves to be a technically feasible and cost-effective alternative to non-outward solutions under various scenarios. Although the expected water level difference exceeds the accepted 1-millimetre threshold as regulated in the Rivierkundig Beoordelingskader (RBK), these higher WLD do not result in immediate failure, thereby confirming the technical feasibility of ODR. The analyses conducted in this thesis are conservative, suggesting that the feasibility of ODR may be underestimated in practice.

The main implication of ODR is a reduction in the functional lifetime of affected dikes. This reduction is limited to only a few years and can be addressed either by accepting it within long-term planning or by applying a thin asphalt layer to reinforce perceived safety. Soil characteristics, subsidence, and climate-related uncertainties have a more substantial influence on failure mechanisms and functional lifetime reduction than the modest water level increases caused by ODR. However, these increases act as a trigger. Still, uncertainties in FLR do not significantly affect the cost-effectiveness of ODR, provided that FLR is accepted. Proper implementation of ODR requires detailed data on the affected dike segments to ensure that no dike fails unexpectedly and that the extent of FLR can be accurately determined.

Riverward reinforcement, where the flood defence system absorbs the hydraulic impact, should be considered when conventional alternatives are hindered by societal resistance, limited space on the landward side, or when creating additional floodplain storage is too complex or costly. ODR offers a cost-effective alternative to construction-based reinforcement, which is currently the preferred approach in such scenarios. Furthermore, a fully soil-based ODR design is more flexible to future policy changes, can be recycled, and demonstrates greater adaptability. It therefore represents a low-regret solution, as highlighted by the Kennisprogramma Zeespiegelstijging's emphasis on flexibility and alternative strategies. Nevertheless, this ODR variant should not be adopted as a standard reinforcement strategy. It reduces floodplain storage capacity, which is not permitted under the RBK without adequate mitigation, and lacks co-benefits such as nature development, which are central to the Room for the River programme. Instead, it should serve as an additional option in complex situations, helping to reduce project complexity today while preserving flexibility for tomorrow.

8 Recommendations

Policy recommendations

It is recommended to formally integrate the outward dike reinforcement (ODR) variant where the flood defence system absorbs hydraulic impact, into the 'Redeneerlijn buitendijks (rivierwaarts) versterken' (line of reasoning for applying outward dike reinforcement, as introduced in Chapter 1 and discussed in Appendix B.3). This strategy should be considered as a follow-up option when ODR with floodplain mitigation proves unfeasible or too costly, but before shifting to construction-based reinforcement. This ensures that ODR remains an supplementary alternative only when inward reinforcement is constrained, aligning with the criteria of the line of reasoning and the situations where ODR is most cost-effective. However, this strategy should only be pursued if it results in significant cost savings, does not conflict with third-party interests, and ensures safe riverbed use, in line with current ODR criteria involving floodplain mitigation. If additional room for the river can be created or floodplain mitigations are feasible, this should remain the preferred solution. After all, the Room for the River initiative goes beyond technical considerations, integrating water quality, nature, and environmental values; benefits that this ODR variant does not offer.

Additionally, it is recommended to update this line of reasoning with the calibrated design graphs and findings from this thesis. These graphs allow for a more accurate approximation of hydraulic impact, thereby enabling more informed decisions on whether or not to implement ODR in specific river sections.

To apply this strategy, the 'Rivierkundig Beoordelingskader' (RBK) should be adjusted where under these specific conditions, a water level difference greater than the 1-mm rule is allowed. A threshold of 2 centimetres is advised for ODR-induced WLD across a full river reach. This value enables a broad range of feasible ODR configurations (as shown by the design graphs in Subsection 3.2.5) and sufficient intervention lengths to ensure cost-effectiveness. A 1-cm threshold is too restrictive due to the steep local water level gradients at the start of interventions, which limit viable intervention lengths.

The new 2-cm threshold remains within acceptable bounds of uncertainty and safety. It is significantly below the identified uncertainty of the design water levels [Warmink et al., 2013] and stability limit of affected dikes. The heave limit state will barely be affected, the assumption of negligible effects on overtopping is valid, and the maximum functional lifetime reduction (FLR) is realistically capped at 5 years, which does not significantly affect cost-effectiveness. The findings of this thesis can be used directly as reference, while still being conservatively interpreted, making the actual impact of the ODR variants less severe than presented.

It is recommended to first clearly define the adaptation endpoint threshold and its calculation method within the RBK. It is advised to use the existing 1-mm rule as this threshold. Additionally, the formulation for calculating adaptation length should be standardised, as small differences in backwater effect modelling can lead to significant variations in tail behaviour and thus adaptation lengths. Simplified models, such as the calibrated 1D model, should apply this same threshold to ensure consistency across assessments.

As for mitigation strategy, it is recommended to accept the FLR. This approach incurs the lowest mitigation costs of the two proposed strategies, is less sensitive to uncertainties governing FLR, and requires minimal practical intervention, thereby reducing potential disturbance. However, this strategy requires thorough assessment of all affected dikes to ensure that no unforeseen failure mechanisms are triggered.

Design recommendations

For design purposes, it is recommended to apply ODR in discrete segments where inward reinforcement is technically or spatially infeasible, or prohibitively expensive, following the updated line of reasoning for ODR. Inward reinforcement remains the preferred strategy and should be pursued wherever feasible. The use of ODR will therefore likely result in discrete applications. These can help reduce local WLD and enable a longer total intervention length without exceeding the updated hydraulic threshold, compared to longer continuous implementation.

Once ODR without floodplain mitigation is identified as a suitable and cost-effective alternative in a specific segment, engineers are advised to explore additional dike sections with similar constraints. This broader exploration enables early identification of segments with similar technical or spatial challenges and should ideally span across multiple reinforcement projects. Such an integrated approach increases the total intervention length, thereby improving the cost-effectiveness of ODR. However, ODR should not be optimised to reach the new 2-cm WLD threshold. Instead, it should be applied only where necessary and beneficial. The goal is not to maximise WLD, but to strategically deploy ODR where it yields the greatest value. Designers are recommended to seek an optimal balance between the use of sheet piles and ODR. In bottlenecks, high WLD may limit the applicability of ODR in other, more favourable locations due to the 2-cm hydraulic limit. A cost-optimal strategy should therefore consider the cumulative hydraulic impact and most cost-effective locations to apply ODR across the entire river reach.
It is recommended to apply ODR mostly in wide floodplains if feasible. In narrow floodplains, it is recommended to apply ODR preferably in areas with high roughness. Bottlenecks (i.e. urban areas directly adjacent to the river) often present relatively high WLD. Nevertheless, ODR may still be viable in these locations, as the greatest benefit can be obtained where spatial pressure exists on the landward side of the dike. It is therefore recommended to consider ODR in such bottlenecks up to 300 metres from the river channel, as applying ODR within the summer bed of the river is not considered feasible.

Finally, designers are advised to carefully balance the amount of outward expansion in riverward reinforcement, selecting a design between the presented Tuimeldijk and the variant with a maximum expansion of 20 metres. The amount should be chosen such that ODR inherently provides sufficient adaptability to accommodate future uncertainties and ensure adequate resistance against potential piping issues without requiring sheet piles. However, the design should not be overly robust, as this would lead to excessive soil costs. Wherever possible, it is recommended to use recycled soil of sufficient quality.

Future research

The findings of this thesis suggest directions for further research to better understand the effects of ODR.

To properly integrate ODR without floodplain mitigation into the line of reasoning for riverward reinforcement, it is recommended to **generalise the hydraulic findings** of this thesis so they can be applied to all river systems in the Netherlands that require assessment under the RBK. This thesis is based on the hypothetical reinforcement of the southern Waal bank, with hydraulic impact analyses focused solely on the Waal River. Generalising the findings would enable comparison of relative ODR effects across different river systems.
Similarly, the current stability assumptions are based on expert heuristics and simplified models. Future research should assess whether the feasibility of ODR remain valid under a 2-cm WLD when **stability is evaluated using more advanced geotechnical modelling**.

It is also recommended to investigate the effect of a 2-centimetre WLD on **morphology, cross-flows, and flow velocity**, and to assess whether these pose critical limitations under current RBK regulations.
Furthermore, additional research is warranted to assess whether the proposed 2-cm WLD threshold would exceed the allowed maximum **discharge distribution change of 20 m^3/s at the Pannerdensche Kop** during peak flow. If this were the case, further analysis is needed to determine the allowable increase in WLD and to identify the downstream river kilometre limit beyond which ODR may be applied in the Waal and Pannerdensche Kanaal.

Lastly, future studies should expand the cost-effectiveness analysis by **integrating engineering and environmental costs, including emissions and material production**, into life-cycle cost assessments. These factors could significantly influence the overall cost-effectiveness of the considered reinforcement alternatives.

Bibliography

Algemene Rekenkamer. Keuzes in het waterveiligheidsbeleid. Technical report, Algemene Rekenkamer, October 2023.

K.D. Berends and F.L.M. Diermanse. Naar eeen conceptuele basis voor omgaan met onzekerheid in stromingsmodellen van Rijkswaterstaat. Technical report, Deltares and Rijkswaterstaat Water, Verkeer en Leefomgeving, December 2021.

K.D. Berends, M.W. Straatsma, J.J. Warmink, and S.J.M.H. Hulscher. Uncertainty quantification of flood mitigation predictions and implications for interventions. *Natural Hazards and Earth System Sciences*, 19(8): 1737–1753, 8 2019. ISSN 16849981. doi: 10.5194/nhess-19-1737-2019.

J. Bessembinder, R. Bintanja, R. Van Dorland, C. Homan, B. Overbeek, F.M. Selten, and P. Siegmund. KNMI'23 klimaatscenario's voor Nederland. https://www.knmi.nl/klimaat/knmi23-klimaatscenario-s, October 2023. Accessed 26 July 2025.

A. Blom. Backwater Curves. Lecture slides, CIEM3000: Base-3 River Systems and Interventions, TU Delft, February 2025a. Topic 06.

A. Blom. Initial Channel Response. Lecture slides, CIEM3000: Base-3 River Systems and Interventions, TU Delft, February 2025b. Topic 1, slide 78.

A. Blom. Initial Channel Response. Lecture slides, CIEM3000: Base-3 River Systems and Interventions, TU Delft, March 2025c. Topic 10.

A. Blom, C. Ylla Arbós, M.K. Chowdhury, A. Doelman, M. Rietkerk, and R.M.J. Schielen. Indications of Ongoing Noise-Tipping of a Bifurcating River System. *Geophysical Research Letters*, 51, November 2024. doi: 10.1029/2024GL111846.

D. Booij. Rivierkundige effectbeoordeling DO. Technical report, Royal HaskoningDHV, May 2022. reference: WATRC_BF6777-115-102_R0095_920854_f3.0.

Boskalis and Royal HaskoningDHV. Deelrapport Rivierkundige beoordeling, Defnitief Ontwerp, October 2022.

A. Camarena Calderon, A.J. Smale, J. Van Nieuwkoop, and J. Morris. Input database for the Bretschneider wave calculations for narrow river areas: In preparation for the WTI-2017 production runs. Technical Report 1209433-000, Deltares, Delft, The Netherlands, 2016.

CROW. SSK in kort bestek. Technical report, CROW - Nationaal kennisplatform voor infrastructuur, verkeer, vervoer en openbare ruimte, 2023. De rol van een kostenraming in de keten.

H.J. De Vriend, M. Kok, J. Pol, and M. Hegnauer. Is there a maximum discharge for the Rhine at Lobith? Technical report, March 2017.

H. De Waal. Basisrapport wbi 2017, versie 1.2. Technical report, Deltares, Delft, The Netherlands, 2018. Deltares, 2018, B.

B. Domhof, K.D. Berends, A. Spruyt, J.J. Warmink, and S.J.M.H. Hulscher. Discharge and location dependency of calibrated main channel roughness: Case study on the River Waal. *E3S Web of Conferences*, 40:06038, January 2018. doi: 10.1051/e3sconf/20184006038.

U. Förster, G. Van den Ham, E.O.F. Calle, and G. Kruse. Zandmeevoerende wellen, March 2012.

C. Geerse. Werkwijze uitintegreren onzekerheden basistoestanden voor Hydra-NL: Afvoeren, meerpeilen, zeewaterstanden en windsnelheden. https://www.knmi.nl/kennis-en-datacentrum/achtergrond/knmi-14-klimaatscenario-s, 2015. Opdrachtgever: RWS-WVL, Lijn in Water, PR3216.10.

Google Earth. Boven-Leeuwen, Netherlands. https://earth.google.com/web/search/Boven-Leeuwen, 2024. Boven-Leeuwen, 51°50'05.6"N, 5°13'51.2"E, elevation unknown. Accessed 3 March 2025.

GraafReinaldalliantie. Plan Duurzaamheid Dijkversterking GoWa - Groeidocument, September 2020.

M. Groenewoud. Veiligheidsanalayse dijkversterking Gorinchem - Waardenburg. Technical report, Waterschap Rivierenland, August 2016.

M. Haasnoot and F.L.M. Diermanse. Analyse van bouwstenen en adaptatiepaden voor aanpassen aan zeespiegelstijging in Nederland. Technical report, Delft, September 2022. Deltares report number: 11208062-005-BGS-0001.

D. Heikens, R. Van de Laar, F. Maters, C. Soree, M. Van den Berg, A. De Wilde, A. Krikken, R. Van Bruchem, and H. Heyl. Dijkversterking Neder-Betuwe: MER dijkversterking Neder-Betuwe. Technical report, Royal HaskoningDHV, September 2022. reference: WATRC_BF6777-114-105_R0097_c2.0.

Hoogwaterbeschermingsprogramma. Life Cycle Costing (LCC) in dijkversterking. https://www.hwbp.nl/kennisbank/life-cycle-costing, 2021. Accessed 23 Augustus 2025.

Hoogwaterbeschermingsprogramma. HWBP langetermijn programmering "Het één doen zonder het andere te laten", November 2023.

Hoogwaterbeschermingsprogramma. Eindadvies Lange Termijn Programmering. March 2024a.

Hoogwaterbeschermingsprogramma. Dijkversterkingsproject Gorinchem - Waardenburg. https://www.hwbp.nl/projecten/overzicht-projecten/rivierenland/gorinchem-waardenburg, October 2024b. Accessed 11 March 2025.

Hoogwaterbeschermingsprogramma. Dijkversterkingsproject Ravenstein - Lith (Meanderende Maas). https://www.hwbp.nl/projecten/overzicht-projecten/rivierenland/gorinchem-waardenburg, October 2024c. Accessed 11 March 2025.

Hoogwaterbeschermingsprogramma. Dijkversterkingsproject Neder-Betuwe. https://www.hwbp.nl/projecten/overzicht-projecten/rivierenland/neder-betuwe, October 2024d. Accessed 18 March 2025.

Hoogwaterbeschermingsprogramma. Dijkversterkingen langs de grote rivieren: Redeneerlijn buitendijks (rivierwaarts) versterken. https://repository.officiele-overheidspublicaties.nl/Bijlagen/TerInzageLegging/2024/til-2024-31632/1/bijlage/5.6factsheethwbp-buitendijksversterken.definitief.pdf, 2024e.

Hoogwaterbeschermingsprogramma. Interactieve projectenkaart. https://www.hoogwaterbeschermings programma.nl/projecten, 2025. Accessed 11 March 2025.

Informatiepunt Leefomgeving (IPLO). Hydra-nl. https://iplo.nl/thema/water/applicaties-modellen/waterveiligheidsmodellen/hydra-nl/, 2025. Accessed 26 July 2025.

S.N. Jonkman, R.E. Jorissen, T. Schweckendiek, and J.P. van den Bos. *Flood Defences: Lecture Notes CIE5314*. Delft University of Technology, 4th edition, April 2021.

A. Klein Tank, J. Beersma, Bessembinder, B. Van den Hurk, and G. Lenderink. KNMI'14-klimaatscenario's voor Nederland. https://www.knmi.nl/kennis-en-datacentrum/achtergrond/knmi-14-klimaatscenario-s, 2015. Accessed 26 July 2025.

J.A. Knappett and R.F. Craig. *Craig's Soil Mechanics: Eighth edition*. CRC Press, 8th edition, January 2012. ISBN 9780415561259. doi: 10.1201/b12841. Publisher Copyright: © 2012 J. A. Knappett.

M. Kok, R. Jongejan, M. Nieuwjaar, and I. Tánczos. Fundamentals of flood protection. Technical report, Rijkswaterstaat and Expertise Netwerk Waterveiligheid, December 2016.

V. Maronier, N. Geurts van Kessel, and H. Nonnekens. Concept MER dijkversterking Gorinchem-Waardenburg: onderbouwing voorkeursalternatief (deel A), September 2018a.

V. Maronier, N. Geurts van Kessel, and H. Nonnekens. Concept MER dijkversterking Gorinchem-Waardenburg: onderbouwing voorkeursalternatief (deel B). https://rhk.maps.arcgis.com/apps/MapJournal/index.html?appid=90e2e2d8468641a38d91056692358d0c section=15, September 2018b. Accessed 18 March 2025.

E. Mosselman. The Dutch Rhine branches in the Anthropocene – Importance of events and seizing of opportunities. *Geomorphology*, 410:108289, 2022. ISSN 0169-555X. doi: https://doi.org/10.1016/j.geomorph.2022.108289.

E. Nilson, C. Viergutz, J. Kruijshoop, M. Nied, A. Handwerk, P. Schmocker-Fackel, S. van Doorn, R.M.J. Schielen, and M. Willkomm. Climate Change Induced Discharge Scenarios for the Rhine Basin. (Technical report no. 297), 2024.

Projectteam Meanderende Maas. Meanderende Maas Terinzage 2024. https://innovaviewer.nl/meanderendemaas-terinzage2024/, May 2024. Accessed 11 March 2025.

J. Rijke and M.J.C.M Hertogh. Inventarisatie beheer- en onderhoudskosten primaire waterkeringen. Definitief rapport 14040/JR/2014.12.31, Triple Bridge, Utrecht, December 2014. Basisgegevens voor LCC binnen het HWBP. In opdracht van Programmadrirectie Hoogwaterbescherming, Ing. Erik Kraaij.

Rijkswaterstaat, Ministerie van Infrastructuur en Waterstaat. Handreiking Beleidslijn Grote Rivieren: Geactualiseerde versie 2019, July 2019. Definitieve versie.

Rijkswaterstaat Waterdienst and Grontmij Nederland B.V. Beeldenboek vegetatiebeheer grote rivieren, 2010. Edited by Hans Jaspers, Perry Cornelissen, Bert Voortman, and Max Schropp. In collaboration with Staatsbosbeheer, Natuurmonumenten, and several regional landscape organisations.

P.P. Sahoo and S.K. Shukla. Taylor's slope stability chart for combined effects of horizontal and vertical seismic coefficients. *Geotechnique*, 69(4):344–345, 2019.

G.J. Schiereck and H.J. Verhagen. *Introduction to Bed, Bank and Shore Protection*. Delft Academic Press / VSSD, Delft, The Netherlands, revised edition edition, 2019. ISBN 978-90-6562-441-3.

T. Shou. A Literature Review on the Net Present Value (NPV) Valuation Method. 01 2022. doi: 10.2991/aebmr.k.220603.135.

SkyGeo. Bodemdalingskaart 2.0. https://bodemdalingskaart-portal.skygeo.com/portal/bodemdalingskaart/v2/viewers/basis/, 2023.

N. Slootjes and H. Van der Most. Achtergronden bij de normering van de primaire waterkeringen in Nederland. Technical report, Ministerie van Infrastructuur en Milieu, DG Ruimte en Water , Directie Waterbeleid en Veiligheid, June 2016.

A.J. Smale. Werkwijzer bepaling Hydraulische Ontwerprandvoorwaarden. Technical report, Deltares, 2018.

E.J. Spierenburg and E.O.F. Calle. Probabilistische gevoeligheidsanalyse heavingontwerpregels. Technical Report CO-370250/51, Grondmechanica Delft, in opdracht van Rijkswaterstaat, Dienst Weg- en Waterbouwkunde, Delft, The Netherlands, June 1998.

A. Spruyt, A. Kosters, E. Van der Deijl, and T. Visser. Handleiding opzet nieuwe D-HYDRO modelschematisatie voor Rijn of Maas. Internal report, 2024. Versie 1.0, 2024.

P.J. Stappers and E. Giaccardi. Research through Design. In *The Encyclopedia of Human-Computer Interaction, 2nd Ed.* Interaction Design Foundation, January 2014. Accessed 28 May 2025.

T. Steward, N. Sivakugan, S.K. Shukla, and B.M. Das. Taylor's Slope Stability Charts Revisited. *International Journal of Geomechanics*, 11(4):348–352, 2011.

Stuurgroep Meanderende Maas. ProjectMER/MER 2e fase. https://innovaviewer.nl/meanderendemaas-terinzage2024/, June 2023a. Accessed 11 March 2025.

Stuurgroep Meanderende Maas. Projectplan Waterwet Dijkversterking Ravenstein - Lith, April 2023b.

R. ’t Hart, H. De Bruijn, and G. De Vries. Fenomenologische beschrijving: Faalmechanismen WTI. Technical Report 1220078-000-GEO-0010-gbh, Deltares, 2016. Delatres, In opdracht van Rijkswaterstaat Water, Verkeer en Leefomgeving.

R. Van de Laar and C. Overgoor. Dijkversterking Neder-Betuwe: Projectplan Waterwet, September 2022. Reference: WATRC_BF6777-114-105_R0096_901971_c2.0; Projectnummer: BF6777.

R. Van der Brugge and R.C. De Winter. Deltascenario’s 2024 - Zicht op Water in Nederland. Technical report, Deltares, April 2024.

J.W. Van der Meer. Wave Run-up and Wave Overtopping at Dikes. Technical report, Technical Advisory Committee on Flood Defence, Delft, May 2002.

J.W. Van der Meer, A. Van Apeldoorn, J.A. Battjes, M.R.A. Van Gent, R. ’t Hart, S.R. Holterman, M. Klein Breteler, A.P. De Looff, M. Van de Paverd, R. Piek, H.M.G.M. Steenbergen, P. Tönjes-Gerrand, J.E. Venema, and J.P. De Waal. Technisch Rapport: Golfoploop en Golfoverslag bij Dijken. Technical report, Technische Adviescommissie voor de Waterkeringen, Delft, May 2002. Projectbegeleiding: Dienst Weg- en Waterbouwkunde.

S. Van Doorn and Rijkswaterstaat Water, Verkeer en Leefomgeving. Rivierkundig Beoordelingskader voor ingrepen in de Grote Rivieren. January 2023.

J.M. Van Esch. WTI 2017: Toetsregel Piping - Piping and Transient Groundwater Flow. Technical Report 1209435-003, Deltares, November 2014.

M. Van Heereveld. Analyse succesfactoren en gebiedspotentieel dijkversterking met gebiedseigen grond. Technical Report WATRC_BG6496-101-102_R0003_413570_f1.0, Royal HaskoningDHV, Nijmegen, The Netherlands, 2020. Project: POV DGG, Status: Definitief, 26 maart 2020.

A. Van Heezik. *Strijd om de rivieren: 200 jaar rivierenbeleid in Nederland*. HNT Historische Producties / Rijkswaterstaat, Den Haag, 2006. ISBN 978-90-806366-3-7.

T. Van Loon, J. Van Minnebruggen, I Officer, K. Van Leeuwen, and F. Van Hemmen. Dijkversterking Waalbandijk Neder-Betuwe: Ruimtelijk kwaliteitskader, January 2018.

M.C.L.M. Van Mierlo, A.C.W.M. Vrouwenvelder, E.O.F. Calle, J. Vrijling, S.N. Jonkman, K. De Bruijn, and A. Weerts. Assessment of flood risk accounting for river system behavior. *International Journal of River Basin Management*, 5:93–104, June 2007. doi: 10.1080/15715124.2007.9635309.

P. Van Veen, M. Niessen, and V. Maronier. Projectplan Waterwet dijkversterking Gorinchem-Waardenburg, October 2020.

J.J. Warmink, M.J. Booij, H. Van der Klis, and S.J.M.H. Hulscher. Uncertainty in water level predictions due to various calibrations. pages 1–18, November 2007. 1st International Conference on Adaptive amp; Integrated Water Management, CAIWA 2007, CAIWA ; Conference date: 12-11-2007 Through 15-11-2007.

J.J. Warmink, M.W. Straatsma, F. Huthoff, M.J. Booij, and S.J.M.H. Hulscher. Uncertainty of design water levels due to combined bed form and vegetation roughness in the Dutch River Waal. *Journal of Flood Risk Management*, 6:302–318, December 2013. doi: 10.1111/jfr3.12014.

Waterschap Aa en Maas, Boskalis, and Royal HaskoningDHV. Dijkontwerp Technische Profielen – Planuitwerking Meanderende Maas, Ravenstein–Lith, October 2022.

Waterschap Rivierenland and Royal HaskoningDHV. Dwarsprofielen Ooij – Dijkversterking Neder-Betuwe, September 2022. Planuitwerkingsfase, documentversie 3.0, projectnummer BF6777-RDH-YP-DG-C-2001.

B.G.H.M. Wichman. Flood Defences and Flood Risk Management. https://www.deltares.nl/en/expertise/areas-of-expertise/floods/flood-defences-and-flood-risk-management, n.d. Accessed 2 March 2025.

Appendices

A

Hydraulic effects of outward dike reinforcement

It is pivotal to understand the hydraulic effects that activities such as the construction of outward dike reinforcement may cause once implemented. If a river maintains consistent characteristics over a long stretch, the water level gradually adjusts toward the equilibrium depth. This is the depth the river attains under uniform flow conditions for a given discharge, resulting from a balance between gravitational and frictional forces. It depends on river width, slope, friction coefficient and discharge, and is defined in Equation A.1 [Blom, 2025a].

$$d_e = \left(\frac{c_f}{i_b} \cdot \frac{q^2}{g}\right)^{1/3} \quad \text{(A.1)} \qquad d_g = \left(\frac{q^2}{g}\right)^{1/3} \quad \text{(A.2)} \qquad q = \frac{Q}{B} \quad \text{(A.3)}$$

River flow is often disturbed by structures, vegetation, or spatial variation, causing the local flow depth d_0 to differ from the new equilibrium depth. The water level then adjusts gradually along a backwater curve (as visualised in Figure 3.3 in Section 3.1.1). The direction and shape of this curve depend on the bed slope and the relation between equilibrium and critical depth, defined by the Froude number $Fr = 1$ (Equation A.2) [Blom, 2025a]. When $d_g > d_e$, the flow is supercritical and effects propagate downstream. When $d_e > d_g$, the flow is subcritical and effects extend upstream [Blom, 2025a]. As rivers like the Waal typically exhibit subcritical flow, interventions primarily affect upstream water levels [Blom, 2025c].

Outward expansion reduce the river width, which increases the specific discharge (Equation A.3 and consequently raises the equilibrium depth. Under subcritical flow conditions, this results in a rise of local and upstream water levels that can extend over a considerable distance. Flood defences are designed for a design high water level [Van Doorn and Rijkswaterstaat Water, Verkeer en Leefomgeving, 2023], events that are relatively short-lived. As a result, the river has little time to undergo morphological adjustment, and only initial responses in flow characteristics may occur. Nevertheless, these initial responses can still cause undesired local effects on the riverbed [Blom, 2025c].

The extent of the backwater effect, which describes the distance required for the flow to return to normal flow, depends on the change in equilibrium depth, the length of the narrowing, and the river slope. When the narrowing is short, the water level can return to its original state a little further upstream. However, substantial water level increases may still propagate upstream and require compensatory measures to maintain flood safety [Van Doorn and Rijkswaterstaat Water, Verkeer en Leefomgeving, 2023]. Equation A.4 [Blom, 2025c], based on Bresse's formulation, provides an empirical estimate of the half-backwater length. This relation shows that steeper slopes result in shorter backwater effects. Although the equation is derived for rectangular channels, it offers a useful approximation. After a distance of four times the half-backwater length, the influence of the backwater curve is no longer observed [Blom, 2025a].

$$L_{1/2} = 0.24\frac{d_e}{i_b}\left(\frac{d_0}{d_e}\right)^{4/3} \quad \text{(A.4)} \qquad u_e = \frac{q}{d_e} = \left(\frac{i_b}{c_f} q g\right)^{1/3} \quad \text{(A.5)} \qquad \tau_b = \rho c_f u^2 \quad \text{(A.6)}$$

Due to narrowing of the river by outward dike reinforcement, both the equilibrium depth and the equilibrium flow velocity increase, as shown in Equation A.5 [Blom, 2025c]. Although both values increase due to narrowing, the effect of the specific discharge dominates the increase of the equilibrium depth, resulting in higher flows [Blom, 2025c]. These elevated velocities lead to increased bed shear stress, as described in Equation A.6 [Blom, 2025a]. If the bed shear stress exceeds the critical Shields parameter, this may trigger bed erosion [Schiereck and Verhagen, 2019]. Outward reinforcement may therefore induce local erosion where the river is narrowed, especially if the bed protection is not designed to withstand these increased stresses.

In addition to increasing bed shear stress, outward dike reinforcement also affect sediment transport and local bed morphology. The Engelund-Hansen relation (Equation A.7) [Blom, 2025c] shows that sediment transport capacity rises with flow velocity. Combined with sediment mass conservation described by Exner (Equation A.8) [Blom, 2025c], changes in sediment flux can lead to either scouring or deposition. This effect results from the mismatch between actual d_0 and equilibrium water depth d_e following a change in river width. While the equilibrium depth increases instantly, the actual depth adjusts gradually along the backwater curve. As a result, the local flow velocity temporarily exceeds its equilibrium value (Equation A.5), which increases the sediment transport capacity. While the suspended sediment concentration remains unchanged, the additional sediment needed to meet this higher capacity is eroded from the bed. As the flow velocity decreases downstream of the narrowing, the sediment transport capacity also declines, causing the excess sediment to be deposited, leading to the formation of a hump [Blom, 2025c]. Such local deposition is considered a negative hydraulic effect due to its impact on navigation, among other things, and requires dredging [Van Doorn and Rijkswaterstaat Water, Verkeer en Leefomgeving, 2023].

$$s = \frac{0.05 c_f^{3/2}}{D(\Delta g)^2} u^5 \qquad \text{(A.7)} \qquad\qquad c_b \frac{\partial z_b}{\partial t} = -\frac{\partial s}{\partial x} \qquad \text{(A.8)}$$

In conclusion, narrowing of the river through outward dike reinforcement during design high water conditions results in elevated upstream water levels due to the backwater effect. This leads to key hydraulic effects, such as increased flow velocities, which may cause local erosion of bed protection, sediment humps, or scouring. The effects depend on the difference and magnitude of the equilibrium depths. These effects are visualised in Figure A.1 and these outcomes reflect the dominant mechanisms, while acknowledging that more detailed or site-specific responses may also play a role.

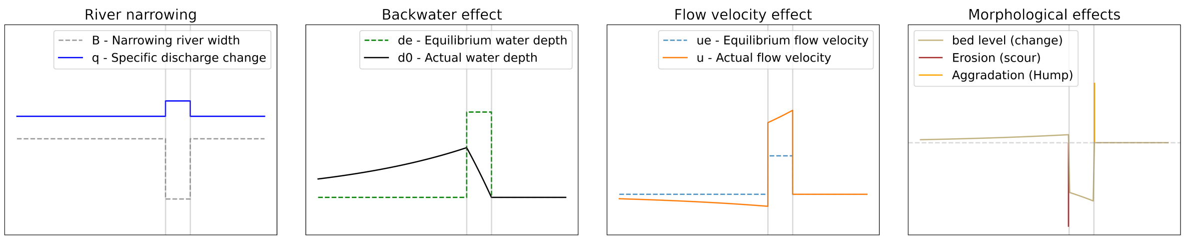

Figure A.1: The hydraulic effects due to ODR visualised

B

Rationale for outward dike reinforcement

Outward dike reinforcement (ODR) offers a technically feasible solution for flood defences. However, it is often not preferred during the decision-making process. This Appendix outlines the relevant policy and assessment framework that must be followed when implementing interventions in the riverbed. Particular attention is given to the one-millimetre rule, which significantly complicates the practical application of ODR. Finally, the section presents the currently applied line of reasoning for the use of outward dike reinforcement, which is formalised in national flood protection policy.

B.1. Beleidslijn Grote Rivieren

As space is not always available directly behind (the inward side of) the dike, as shown in several case studies in Appendix C, reinforcement on the outer side could offer a promising alternative. Strengthening at the riverside of the dike instead of the inner side may significantly reduce costs, shorten development time, and simplify implementation [Rijkswaterstaat, Ministerie van Infrastructuur en Waterstaat, 2019]. However, as discussed in Appendix A, narrowing the river corridor can lead to increased water levels and negative hydraulic effects. This is particularly critical during extreme flood events, where even small reductions in river discharge capacity can pose serious risks to flood safety [Hoogwaterbeschermingsprogramma, 2024e]. While a single riverward dike may have limited impact, current policy assumes that the cumulative effects of ODR could be significant.

Since the high water events of 1993 and 1995 in the Netherlands, it became clear that maintaining sufficient space for rivers was essential. At the same time, societal pressure against the continued enlargement of dikes, due to the neglect of stakeholder interests, prompted a shift from solely reinforcing dikes to creating more room for the river [Van Heezik, 2006, Rijkswaterstaat, Ministerie van Infrastructuur en Waterstaat, 2019]. This principle has been incorporated into the Room for the River policy, which was replaced in 2006 by the 'Beleidslijn Grote Rivieren' (Policy Guideline for the Major Rivers, BGR). The BGR sets out regulatory conditions, in accordance with the Water Act and the Spatial Planning and Environmental Acts, for assessing interventions and modifications within the floodplains and river retention regime under the control of the state [Rijkswaterstaat, Ministerie van Infrastructuur en Waterstaat, 2019, Van Doorn and Rijkswaterstaat Water, Verkeer en Leefomgeving, 2023].

Because the major rivers in the Netherlands are fully confined by flood defences, all high water must be conveyed through the riverbed to prevent overflowing. Obstacles within the riverbed create turbulence, which increases water levels and reduces the river's discharge capacity. With discharges expected to increase [Nilson et al., 2024], pressure on the river system will grow. Therefore, to safeguard flow capacity, the BGR policy restricts construction in floodplains and riverbeds that could hinder the flow regime [Rijkswaterstaat, Ministerie van Infrastructuur en Waterstaat, 2019].

Under the BGR, all activities in the riverbed require a permit and are assessed based on their hydraulic effects and spatial placement. A key condition is that interventions must serve a river-related function and must not negatively affect the flow or flood retention regime. In addition, developments that obstruct or limit future opportunities to create more room for the river are not permitted [Van Doorn and Rijkswaterstaat Water, Verkeer en Leefomgeving, 2023]. Permitted functions include flood defences such as outward dike reinforcement, navigation infrastructure, ecological restoration, water quality improvements, and riverbed maintenance. Non-river-related activities are only allowed in exceptional cases, when no alternatives exist and no adverse effects occur, or when they demonstrably improve the flow regime. Hydraulic impacts resulting from interventions are assessed by Rijkswaterstaat as part of the permitting process, based on the criteria defined in the 'Rivierkundig Beoordelingskader voor ingrepen in de Grote Rivieren' (River Assessment Framework for Interventions in Major Rivers (RBK)). [Rijkswaterstaat, Ministerie van Infrastructuur en Waterstaat, 2019].

B.2. Rivierkundig Beoordelingskader van de grote rivieren

'Het Rivierkundig Beoordelingskader voor ingrepen in de Grote Rivieren' (River Assessment Framework for Interventions in Major Rivers (RBK)) is used in the permitting process for all interventions in the major rivers in the Netherlands, to evaluate the impact of interventions in these riverbeds in an 'unambiguous and uniform manner' [Van Doorn and Rijkswaterstaat Water, Verkeer en Leefomgeving, 2023]. However, detailed hydraulic assessment is only required after consultation with Rijkswaterstaat if significant hydraulic effects are expected. If an assessment compliant with the RBK shows that an intervention negatively affects the flow regime, compensatory measures are required to obtain a permit, or the permit may be denied altogether. In addition to ensuring the safe discharge of water, sediment, and ice, Rijkswaterstaat also evaluates whether third parties are adversely affected, either through the RBK or other applicable policies and frameworks [Van Doorn and Rijkswaterstaat Water, Verkeer en Leefomgeving, 2023].

Since 2023, the sixth version of the RBK has been in use. Assessments are carried out using the D-Hydro Suite flow model, the D-Fast Morphological Impact Model, and the D-Fast Bank Erosion Model [Van Doorn and Rijkswaterstaat Water, Verkeer en Leefomgeving, 2023]. The RBK criteria are organised into three main themes:

- Flood safety
- Disturbance or damage resulting from hydraulic effects
- Morphological effects

A further distinction is made based on the characteristics of the affected river area, primarily determined by the functioning of the floodplains. This depends on whether the floodplains exhibit an active flow regime, primarily driven by river discharge, or function as a flood storage regime, influenced by tides, wind, and sea or lake levels. The evaluation criteria of an intervention differ for each of the three themes vary depending on the area of which is assessed. These are the Rhine branches, the Meuse, the Rhine-Meuse estuary, and the IJssel-Vechtdelta [Van Doorn and Rijkswaterstaat Water, Verkeer en Leefomgeving, 2023]. For these four distinguishing areas, the criteria are elaborated and summarised separately in the RBK.

This thesis primarily focuses on the effects of ODR on flood safety in the Waal River. Therefore, only these criteria will be elaborated on. Firstly, it is stated in the RBK that all interventions in the riverbed must be evaluated based on the criteria in the Water and Planning & Environmental Act. If that is acknowledged, the following criteria to assure flood safety must be met for interventions in the riverbed to be allowed [Van Doorn and Rijkswaterstaat Water, Verkeer en Leefomgeving, 2023]:

1. **The one-millimetre rule**
   An intervention may not cause a flood reference level elevation in the middle axis of the river, compared to the reference case where this intervention will not be realised. If activity in the riverbed does result in a higher water level elevations, this must be compensated, so the effects of the backwater curve are mitigated and not observed. After an intervention, the maximum allowable water level rise in the axis of the river that will be accepted is 1 millimetre. *The one-millimetre rule* is used as a buffer for model uncertainty [Van Doorn and Rijkswaterstaat Water, Verkeer en Leefomgeving, 2023].

   The one-millimetre rule may be omitted if this stems from a compensation measure. After a river widening measure to compensate for water level rise, there will always be a downstream peak which is almost impossible to avoid. This will be accepted if the following three conditions are met [Van Doorn and Rijkswaterstaat Water, Verkeer en Leefomgeving, 2023, Boskalis and Royal HaskoningDHV, 2022]:

- The designs causing the downstream peak and the conservation of water level reduction have been optimised.
- The measure results in a significant net water level reduction over an extended reach.
- Water elevation will be avoided for third parties as good as possible.

2. Compensation of storage capacity
   Activities or interventions in the riverbed change the storage capacity and flow velocities of the river during high flows, also outside the middle axis of the river. Even in low-flow areas, the reduction of storage capacity could lead to high water levels upstream. The effects are dependent on the flow conditions, which determine the requirement for compensation. After an intervention, the storage capacity lost must be completely restored or increased in the same riverbed. This must be realised on the same elevation level, so the same water volume can be retained during high water flows [Van Doorn and Rijkswaterstaat Water, Verkeer en Leefomgeving, 2023].

3. Discharge distribution at Pannerdensche Kop at high flows
   Due to backwater effects near the Pannerdensche Kop, where the Rhine splits into the Waal river and the Pannerdensche channel, the water distribution could be changed. Measures are not allowed to change the discharge distribution at this bifurcation, while a different discharge distribution could lead to unaccounted water levels. For the design discharges, a maximum discharge change of 5 m^3/s is accepted. For flood design discharges, this maximum change is increased till 20 m^3/s [Van Doorn and Rijkswaterstaat Water, Verkeer en Leefomgeving, 2023].

4. Ensuring ice discharge
   Ice damns could case floods, because these halt the flow of water. Therefore, it is required that ice sheets can flow towards the sea without hindrance. Therefore, design guidelines should be followed to promote proper ice discharge [Van Doorn and Rijkswaterstaat Water, Verkeer en Leefomgeving, 2023].

   - For flows ranging from bankfull to those with a return period of 1/75 years, the Froude number must be higher than 0.08.
   - Local changes in normal river width must be avoided.
   - Avoid shallow depthness in the low-flow channel

ODR reduces the effective river cross-section, leading to higher water levels and a loss of flood storage and discharge capacity, as mentioned in Section A. The extent of these impacts depends heavily on the specific location of implementation. As a result, outward dike reinforcement is not always a suitable design alternative under the criteria set out in the RBK. To address this, the state has developed a rationale for assessing whether outward dike reinforcement can be considered a viable design option [Van Doorn and Rijkswaterstaat Water, Verkeer en Leefomgeving, 2023].

B.3. Redeneerlijn buitendijks (rivierwaarts) versterken

Although outward dike reinforcement complicates the design process due to the assessment criteria set by the RBK, it should still be considered in cases where space or public support is lacking on the inner side of the dike, or where technical complexity or high construction costs make other solutions unfeasible [Algemene Rekenkamer, 2023, Hoogwaterbeschermingsprogramma, 2024e]. A line of reasoning for applying outward dike reinforcement ('Redeneerlijn buitendijks (rivierwaarts) versterken') has been developed by the state, Flood Protection Programme and the water boards, aligned with both the BGR and RBK, which must be applied to all projects under the Hoogwaterbeschermingsprogramma (HWBP). This framework for considering ODR sets out a number of criteria that must be met [Hoogwaterbeschermingsprogramma, 2024e].

The line of reasoning first states that outward dike reinforcement should be avoided whenever possible. Priority should be given to inner dike reinforcement. However, inner dike expansion is considered unsuitable if:

- The cost of inner dike expansion is significantly greater than what the costs of riverward dikes would be.
- Inner dike expansion results in unfeasible realisation or with too extreme realisation hazards.
- If societal values experience too much disturbance.

If the criteria for inner dike reinforcement are not met, an implementation of ODR may be considered. However, this may only be considered if:

- The location of the dike reinforcement is not located in a hydraulic unfavourable spot.
- The implementation of riverward dikes do not interrupt the safe usage of the riverbed.

A location is considered unfavourable when the river is already relatively narrow or characterised by high flow velocities, and further narrowing would lead to significant hydraulic impacts or increases in water levels [Hoogwaterbeschermingsprogramma, 2024e], as explained in Section A. If a site meets these criteria, the RBK assessment framework must still be applied to evaluate the suitability of outward dike reinforcement. In particular, the water level effects, as defined under point 1 of the RBK criteria, must be compensated. The line of reasoning also states that the cumulative water level effects of the outward expansion has to be visualised, with and without compensation measures. The mitigation measures for the water level effects are not required to take place at the same time or location as the dike reinforcement, but it must be specified when and where they will be implemented. Preferably, these measures are combined with efforts to enhance ecological value, create additional room for the river, or improve conditions for the navigation sector [Hoogwaterbeschermingsprogramma, 2024e].

The criteria set out in the line of reasoning for outward dike reinforcement and the RBK are relatively strict. As a result, ODR is often disregarded as a reinforcement solution, since their application can turn a seemingly straightforward dike reinforcement project into a complex endeavour due to the need to incorporate the required mitigation measures (Van der Berg, M,. personal contact, February 2025). Therefore, in practice, flood defence engineers are reluctant to utilise outward expansion as a solution, and disregard these without a detailed hydraulic assessment [Kok et al., 2016]. As shown in the case studies of previous reinforcement projects in Section C, only dikes located in the lee of the flow or at a considerable distance from the riverbed were considered suitable for riverward dikes. Other dike segments were excluded based on their location within the river system.

C
Case study of dike reinforcement projects

This Appendix examines previous Flood Protection Programme projects to establish the requirements, objectives, and constraints for dike reinforcement initiatives. This case study focusses on the projects 'Meanderende Maas' (Meandering Meuse River), Gorinchem-Waardenburg (GoWa) and the Neder-Betuwe reinforcement project, whose designs were approved and whose implementation started in 2024, 2021 and 2024, respectively [Hoogwaterbeschermingsprogramma, 2024b,c,d].

C.1. Meanderende Maas

The 'Meanderende Maas' is a dike reinforcement project from the Flood Protection Programme (HWBP) that spans from Ravenstein to Lith on the southern banks of the Meuse. It was claimed as one of the 14 most urgent stretches for reinforcement by HWBP [Stuurgroep Meanderende Maas, 2023b]. A total of 26.6 kilometres of dikes, divided into 11 segments, were found to be non-compliant with safety standards for crest height, stability, and piping [Stuurgroep Meanderende Maas, 2023a]. In parallel, water level reduction measures were considered and implemented in the adjacent floodplains to enhance overall flood safety, in line with the maximum allowable flooding probability of 1 in 10,000 per year [Slootjes and Van der Most, 2016]. The project is currently in the realisation phase, with all dike reinforcements scheduled for completion by 2026 [Projectteam Meanderende Maas, 2024].

Design visions

The dike segments are divided based on the characteristics of the dike and the surrounding environment, so designs can be optimised for their direct surroundings. Furthermore, these segments are divided even further into a combined 60 dike units, based on their ground level, talus and berms [Stuurgroep Meanderende Maas, 2023b]. The foundation of each new design for the dike units stems from the motto 'unity through diversity' [Stuurgroep Meanderende Maas, 2023b], where new dikes have to be assimilated into the current dike topology. The reasoning behind this vision is that the dikes improve the aesthetic character and readability of the environment.

The inner side of the dike is characterised by four historic villages, Megen, Macharen, Oijen, and Lithoijen, that originated on the first river dunes. These villages hold historical significance, are situated at the inner toe of the dike, and maintain a dynamic connection with it. This historical value of the villages assimilated into the environment, had to be enhanced: removal was not a consideration [Stuurgroep Meanderende Maas, 2023b]. Therefore, the dike routes had to be adapted to these villages while also aligning with the historical topology. The dike designs were required to avoid any negative impact on societal values. Within this category not only properties and historical buildings were included, but also distinctive tree lines, characteristic landscapes, scenic views, and functional features. Only the spaces of front yards and agricultural fields without much societal value were allowed to be taken into consideration for designs. However, due to the possible negative hydraulic effects of outward dike reinforcement, an intervention at the inner side of the dike was still taken into consideration. Also, soil-based designs were favoured, but only if it would fit in the area. [Stuurgroep Meanderende Maas, 2023b,a]. Often this was not possible or the great amount of soil used for the failure criteria did not meet the desired spatial quality [Stuurgroep Meanderende Maas, 2023a].

Design choices

The final dike designs were heavily influenced by the societal values outlined in the design vision.
On the inner side, expansion was restricted in all segments 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, and 11 due to the presence of buildings and infrastructure, with just a few exceptions in certain dike units. As a result, reinforcement relied on heave and stability screens to maintain flood protection while preserving existing structures. In some cases, stability berms were applied where space allowed, but inward expansion was generally avoided to protect historical and societal assets. Although inward expansion was initially preferred, it was rarely feasible in practice [Stuurgroep Meanderende Maas, 2023a].

Outward dike expansions were generally restricted to 5 to 10 meters into the floodplains but were in some instances feasible, as seen in segments 2, 3, 5, 6, 8, 9, and 10 and visible in Figure C.1. However, outward dike reinforcements (ODR) were avoided where significant hydraulic effects could occur if the current dikes were too close to the river, as seen in units of dike segments 5, 6, 7, 8, 9 and 11. Besides hydraulic effects, values to nature also crossed out outward expansion as an option. For example in a part of segment 3, where a gutter was marked for its contribution to biodiversity, so a heave screen was chosen. Tree lines played a key role in segments 2 and 5. In segment 2 this required partial relocation of the dike into the floodplains to preserve these landscape features. In segment 9, compensatory measures were introduced to balance the river's storage capacity, ensuring flood safety was not compromised. Similarly, in segment 10, where outward dike expansion was applied, adjustments were made to protect existing infrastructure and minimize interference with the river system, leading to a combination of heave screens and outer stability berms [Stuurgroep Meanderende Maas, 2023a, Projectteam Meanderende Maas, 2024].

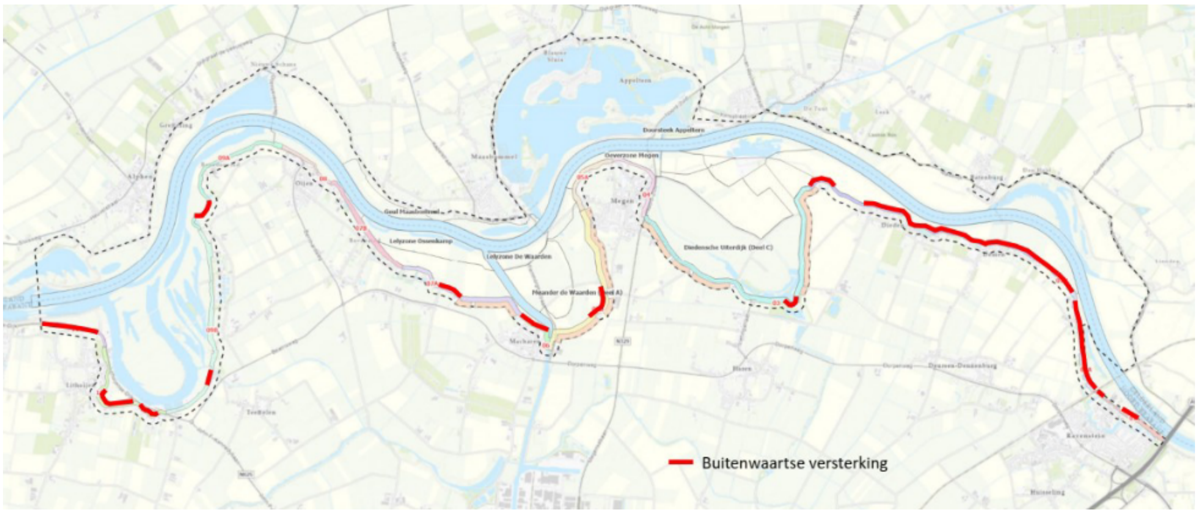

Figure C.1: Locations were riverward reinforcement was chosen as a design solution (in red) in the Meandering Meuse project [Stuurgroep Meanderende Maas, 2023b].

Overall, the final dike designs prioritised integration with the surrounding area and the conservation of buildings and societal values, rather than simply maximising available space and choosing soil-based solutions. Outward dike reinforcement was only considered when hydraulic effects were deemed acceptable or mitigated, often determined by the distance between the dike and the river [Stuurgroep Meanderende Maas, 2023a].

Gastvrije Waaldijk

The northern banks of the Waal had to be reinforced, while many trajectories did not meet the required safety standard defined by the new probability of flooding implemented in 2017, as mentioned in Chapter 1. This resulted in several reinforcement projects between Gorinchem and Nijmegen. Although the reinforcement of the northern banks was divided into five major projects, the cooperating municipalities and the water boards took the opportunity to combine all the projects and create a dike with a similar character. The dike should be inviting for recreation and safe for cyclists and pedestrians while providing stunning views and flower-filled embankments. Particular attention was paid to traffic safety criteria and the visibility on the dike, by implementing slow crossroads and treating all traffic modes as equal, thus making the dikes more accessible and safe for slow traffic. This umbrella project was called the 'Gastvrije Waaldijk' (Welcoming Waaldijk) [Van Veen et al., 2020, Van de Laar and Overgoor, 2022]. The HWBP projects GoWa and Neder-Betuwe are integrated into the Gastvrije Waaldijk and therefore share design visions, while simultaneously differing in focus

C.2. Gorinchem-Waardenburg

Between Gorinchem and Waardenburg, a stretch of 23 kilometres had to be reinforced with a maximum flood probability of 1/10,000 per year [Slootjes and Van der Most, 2016]. This section, like the Meandering Meuse project, was identified by the HWBP as one of the most urgent sections to be reinforced, while along the entire section, the dikes failed for 3 or all 4 main failure criteria, as defined in Chapter 4. The dike section is divided into 14 segments and 51 dike units, determined by soil characteristics, spatial arrangement, and adjacent structures. The reach is designed for a technical lifetime of at least 50 years and is currently in the implementation phase. The project combines inward expansion, ODR and construction measures. The project is under the responsibility of the Waterboard 'Rivierenland' (WSRL) [Van Veen et al., 2020].

Design visions

WSRL aimed to integrate multiple opportunities into the dike reinforcement design, going beyond merely ensuring safety against all failure mechanisms. In addition to aligning with the Gastvrije Waaldijk vision, WSRL prioritised local liveability. The new dike must fit within the characteristic river landscape, preserving its historical trajectory, sharp corners, and bends to reflect past struggles with water. Continuity and recognisability are essential, both for the landscape and traffic safety. Monumental buildings, historical quay walls, boulevards, and tree lines must be maintained. The dike should remain a distinct barrier between the river and the hinterland, retaining its steep talus without high berms that would alter its perception. These objectives must be balanced with sustainable design principles set by WSRL [Van Veen et al., 2020].

WSRL prioritised sustainability in the design process, aiming to showcase an inspiring and circular approach. Soil-based solutions were preferred over structural constructions, with locally sourced materials and professional reuse of old dike materials. Minimal intervention was favoured unless absolutely necessary [GraafReinaldalliantie, 2020]. The designs had to be compact, integrating the dike into private lands or creating value directly behind it. Inner dike solutions were prioritised to prevent negative hydraulic effects, while outward dike expansion was only considered when unavoidable and with mitigated impact. This design issue was turned into an integrated approach to create new ecological value in floodplains [Van Veen et al., 2020].

Design choices

A structured approach was taken for dike reinforcement, prioritising inner dike interventions first, followed by outward dike reinforcement, and lastly constructions, based on an opportunity assessment and in line with approach 'Redeneerlijn buitendijks (rivierwaarts) versterken', as discussed in Appendix B. Alternatives were only replaced when a segment was classified as a 'no go' due to monumental buildings, property constraints, or extreme hydraulic effects on the Waal. If construction proved unfeasible, the assessment reverted to inner dike solutions, refining the design with detailed emplacement around local buildings, such as berms in gardens or tailored adjustments discussed with stakeholders [Maronier et al., 2018a].
Initial assessments considered separate alternatives, where constructions affected the fewest buildings (30), inner dike solutions impacted 161 and outer dike solutions affected 80. However, the final design combined all three strategies to balance feasibility, hydraulic effects, and stakeholder concerns. While outward dike expansions altered sightliness, they were often preferred over inner dike reinforcements, while these posed a greater risk of a loss of community bond. [Van Veen et al., 2020, Maronier et al., 2018a].

Although preserving buildings was a priority, three were ultimately demolished where no viable alternative was found. The final design included 7 kilometres of inner dike reinforcement, 10 kilometres of outward dike reinforcement and 6 kilometres of construction-based reinforcement [Van Veen et al., 2020]. This is visualised in Figure C.2. Constructions were necessary in cases where inner dike solutions affected too many buildings, while ODR was ruled out in some segments due to hydraulic concerns or restrictions from monumental structures and nature preservation. Additionally, more detailed research confirmed that the soil under old dike crest had strengthened due to extended settlement, allowing for slimmer outward dike reinforcement than initially planned [GraafReinaldalliantie, 2020].

In segment 1, the presence of a Natura-2000 area made ODR unfeasible, necessitating an alternative solution. A similar issue arose in segment 7g, where construction was chosen instead of ODR to preserve the existing trajectory, and visibility and avoid disrupting social cohesion. In contrast, in segment 12g, ODR was preferred precisely to maintain local cohesion. Several segments, including 4a, 6, 7a, 7i, 7j, and 12a, were deemed unsuitable for riverward expansion due to hydraulic constraints. However, in some cases, these constraints were reconsidered in favour of saving properties. In segment 5, a water level difference of 3 millimetres was accepted to mitigate the negative impact on a dozen properties, as the area was located in the lee of the river. In segment 8, despite a water level increase of 8 millimetres, ODR was chosen to preserve 15 houses and historical features, whereas, in segment 12a, a singificant lower backwater effect was already considered unacceptable, leading to the selection of a construction alternative. The presence of a small floodplain behind this segment may have influenced this stricter assessment. To mitigate a total 2.5 cm of water level differences, four new floodplains were created, though primarily to compensate for lost Natura-2000 areas [Maronier et al., 2018a, Van Veen et al., 2020].

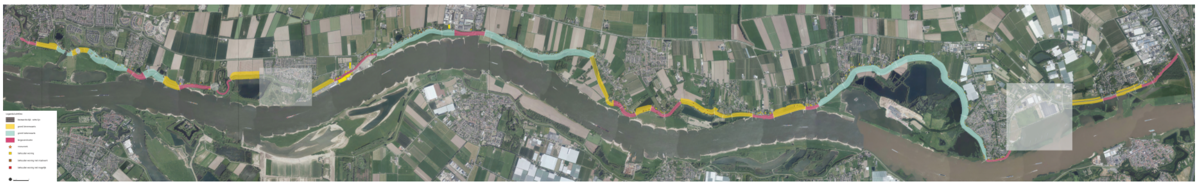

Figure C.2: Chosen designs for the Groningen-Waardenburg reinforcement project [Maronier et al., 2018b]. Yellow: Inward dike reinforcement. Light blue: Outward dike reinforcement. Red: Construction

C.3. Neder-Betuwe

The Neder-Betuwe dike trajectory, stretching 20.2 kilometers between Tiel and Wolferen, has been found to be entirely susceptible to failure due to piping. Additionally, most segments of the dike do not meet the updated 2017 safety standards for slope stability and crest height. Like all northern dike sections along the Waal River, it is required to comply with a maximum annual flooding probability of 1 in 10,000 [Slootjes and Van der Most, 2016]. The section is divided into nine dike segments and 31 dike units, based on existing dike topography, soil characteristics, and key landscape features. The area behind the dike is characterised by fruit orchards, tree cultivation, remnants of the Waal's historical flow, and old settlements dating back to the Roman era. The newly designed soil dikes have a technical lifespan of 50 years, extending until 2075, and are currently being prepared for construction [Van de Laar and Overgoor, 2022].

Design visions

The main goal of the water board Waterschap Rivierenland (WSRL) was to properly place the dikes into the surrounding environment and to keep dike segments that contribute to social cohesion. To achieve this, a spatial quality framework (Ruimtelijk KwaliteitsKader) was developed to be used during the design process [Van de Laar and Overgoor, 2022]. From this framework, the dike designs had to follow certain principles.

First, the dike trajectory should be continuous and flowing. This must be done be assuring slim dikes, with steep talus of 1:3, a single and slim crest and berms that are significantly lower than the crest level. Furthermore, the current flowing and bending pattern of the dike trajectory should be maintained or exaggerated, instead of creating a straight trajectory. This approach was intentionally chosen to maintain a consistent dike character, aligning with the adjacent dike trajectories to the east and west of Neder-Betuwe, including the GoWa dike trajectory. The integration of the dike into its surrounding landscape must be preserved.

This means not only safeguarding historical fields and villages, but also retaining characteristic features such as wet outer toes or providing suitable compensations where preservation is not possible. The dike should blend harmoniously with its environment, avoiding any disruption or fragmentation of the landscape. The dike should be positioned as close as possible to existing land use, allowing the inner berms to be utilised by neighbouring agricultural businesses. This prevents the creation of unnecessary buffer zones and supports a dynamic, functional dike landscape. Beyond its practical role, this approach also reinforces the historical narrative of the region's enduring relationship with water. As a vision it was proposed to not only maintain the historical aspect around the dike, but also to return lost values by building on the dike crests. However, this vision was not shared with stakeholders. Lastly, the same design visions as for the umbrella project Gastvrije Waaldijk are maintained, to keep the dike accessible for recreation [Van Loon et al., 2018].

The principles of the spatial quality framework were incorporated into a landscape vision, serving as a benchmark to evaluate alternatives such as inward expansion, outward dike reinforcement, or construction. The vision emphasized amplifying the historical character and use of the dike, avoiding demolition of buildings, and preserving the surrounding landscape as much as possible. Enhancements included adding recreational pathways and natural elements that highlight the dike's historical significance. These principles aimed to maximize the experience of the dike, the Waal's scenery, and the Betuwe's landscape on both sides. Sustainability was also considered during the process [Van de Laar and Overgoor, 2022].

Design choices

The same design approach used for the GoWa trajectory was applied here, prioritising inner dike expansion, followed by outward dike reinforcement. Only when neither of these options proved feasible was the use of engineered constructions considered. A detailed soil investigation was also conducted to determine the strength of the existing dike material. This allowed the design to make use of the available soil-bearing capacity, enabling slimmer and less conservative reinforcement solutions where possible. Outward dike reinforcement was often ruled out in areas where cumulative backwater effects approached the 1-millimetre threshold (as discussed in Appendix B). This was the case, for example, in the Wely dike segment, where inner dike reinforcement and engineered constructions were implemented instead. In several instances, constructions were chosen over inner berms even when the latter was technically viable. Particularly when constructions better aligned with the existing dike trajectory or allowed for the preservation of private properties, as seen in the eastern part of the city Dodewaard and central of De Snor [Heikens et al., 2022].

Where inner dike berms were feasible but would have compromised buildings or gardens, designs shifted toward outward expansion if hydraulic effects were minimal or non-existent, such as in parts of Dodewaard and unit DT070 of De Snor. Similarly, in Eldik and IJzendoorn, outward dike reinforcement was preferred to avoid impacts on inner-side properties and because the wide floodplain allowed for such designs without exceeding hydraulic thresholds. At the first dike unit of segment Ochten, a construction was chosen over expandable dikes although more expensive, to preserve natural and archaeological values in the floodplain. Inner dike solutions were also avoided here due to concerns about social cohesion. In the denser urban area of segment Ochten, the design choice was a construction, as both heritage buildings on the inner side and limited floodplain space on the outer side made other options unviable [Heikens et al., 2022].

Wherever spatial or environmental constraints were minimal, simpler inner dike reinforcements were applied. This occurred in Echteld and extended into the adjacent unit DT172 in Ooij. However, in DT172, outward expansion was ruled out due to unacceptable hydraulic effects. Further downstream, where the floodplain widened, outward dike reinforcement became feasible again and was chosen to preserve trees, buildings, and a road behind the dike. Finally, in the highly constrained area of dike segment Kanaaldijk near the Amsterdam Rijnkanaal, only inner dike solutions were possible due to space limitations [Heikens et al., 2022].

Figure C.3: Locations of outward dike reinforcement as a alternative (red) in the Neder-Betuwe reinforcement project [Booij, 2022]. Blue: Inward expansion or preserving current profile

Dike reinforcement projects takeaways

Across all three reinforcement projects, inner dike reinforcement was the preferred intervention, but its feasibility was often limited by buildings, historical values, or other land use behind the dike. In the Meandering Meuse project, this approach was rarely applied. In both the GoWa and Neder-Betuwe projects, the design approach extended beyond preserving values on the inner side of the dike, but it also required maintaining close proximity to existing villages. Where relocation was unavoidable, the areas directly behind the dike had to be reoccupied to preserve their societal function. While this vision supports a lively and historically rooted dike landscape, it also introduces limitations. By restricting future inner dike reinforcements, it reduces adaptability and complicates the implementation of additional flood defence layers, as discussed in Chapter 1. If buildings are not allowed to interact with or adapt to the dike, entire areas behind the dike become difficult to modify, further constraining long-term spatial and infrastructural flexibility.

While outward dike reinforcement serves as a second alternative, it is frequently dismissed if the dike is too close to the river, often without a detailed assessment of the hydraulic feasibility. In the Neder-Betuwe project, the 1-millimetre backwater threshold made outward expansion difficult to justify in narrow river sections or upstream areas. The same applied to the other two projects, although GoWa occasionally prioritised outward dike reinforcement over constructions, even with significant backwater effects. In such cases, newly created floodplains were used to mitigate these effects while preserving natural values. The Meandering Meuse project also introduced lowered and additional floodplains to increase storage capacity, but this rarely influenced the decision to implement outward expansion. Across all three projects, outward dike reinforcement was primarily considered when a floodplain was present or when these were located in the lee of the river, as this helped mitigate backwater effects. However, this restriction significantly limited the application, even when construction was feasible and hydraulic effects could have been mitigated.

As a result, inner dike reinforcement remains the preferred option, but spatial constraints, particularly the presence of buildings, often make it unfeasible. Outward dike reinforcement is the next logical choice, yet they are frequently disregarded due to water level increases. Furthermore, despite being technically feasible, cost-effective, and adaptable, outward expansion is often de-prioritised in favour of maintaining a flowing dike trajectory and continuity, landscape and scenic aesthetics, historical values and mostly maintaining buildings, prioritising these aspects over purely cost-optimal or technically efficient solutions. In such cases, more expensive construction designs are chosen. Although, in some instances, extending construction solutions between dike sections has proven to be the more cost-effective approach.

D
Adaptation length endpoint

Outward dike reinforcement increases the equilibrium water depth, resulting in backwater effects along the length of the intervention. Beyond this zone, the water level gradually returns to its original equilibrium depth. The distance over which this transition occurs is referred to as the adaptation length.

The spatial extent of the D-Hydro simulations was insufficient to determine the full adaptation length for the applied reinforcements (see Appendix G). Therefore, a simplified one-dimensional compound channel model was used to analyse backwater effects. This model applies an empirical fit to the Bresse approximation, as described in Subsection 3.1.1.
According to Bresse's backwater theory, backwater effects become negligible beyond four times the half backwater length [Blom, 2025a]. Alternatively, the adaptation length can be defined using an absolute threshold criterion, where the water level difference relative to the original equilibrium depth has decreased to lower than the applied threshold. Applying both methods to the Waal, using calibrated D-Hydro parameters, revealed significant differences in the resulting adaptation lengths, as shown in Figure D.1.

The upper panel shows backwater effects for various intervention lengths, with either relative endpoints (orange) based on Bresse's backwater theory or absolute endpoints (purple and red). The absolute threshold-based endpoints exhibit greater variability and more closely resemble the simulated adaptation behaviour observed with the extrapolated D-Hydro simulations (Appendix G. The theoretical endpoints, by contrast, deviate significantly.

The lower panel plots adaptation lengths, measured from the point of maximum water level rise to the endpoint, against intervention length. The absolute threshold-based method shows a clear increasing trend with longer interventions, consistent with the expected behaviour of a backwater curve. The theoretical method does not reflect this relationship and seems not to be affected by a water level increase of a couple of centimetres.

Since the 'Rivierkundig Beoordelingskader' applies a strict 1-mm criterion, it would be illogical to adopt a theoretical endpoint that fails to respond to changes in intervention length and water level. Moreover, for most relative threshold-based endpoints, the water level remains above 1 mm. While the absolute threshold method may be conservative, it better reflects the expected hydraulic behaviour and is therefore used in this study to calibrate the adaptation length factor between the 1D model and D-Hydro results, and for further assessments using the 1D model.

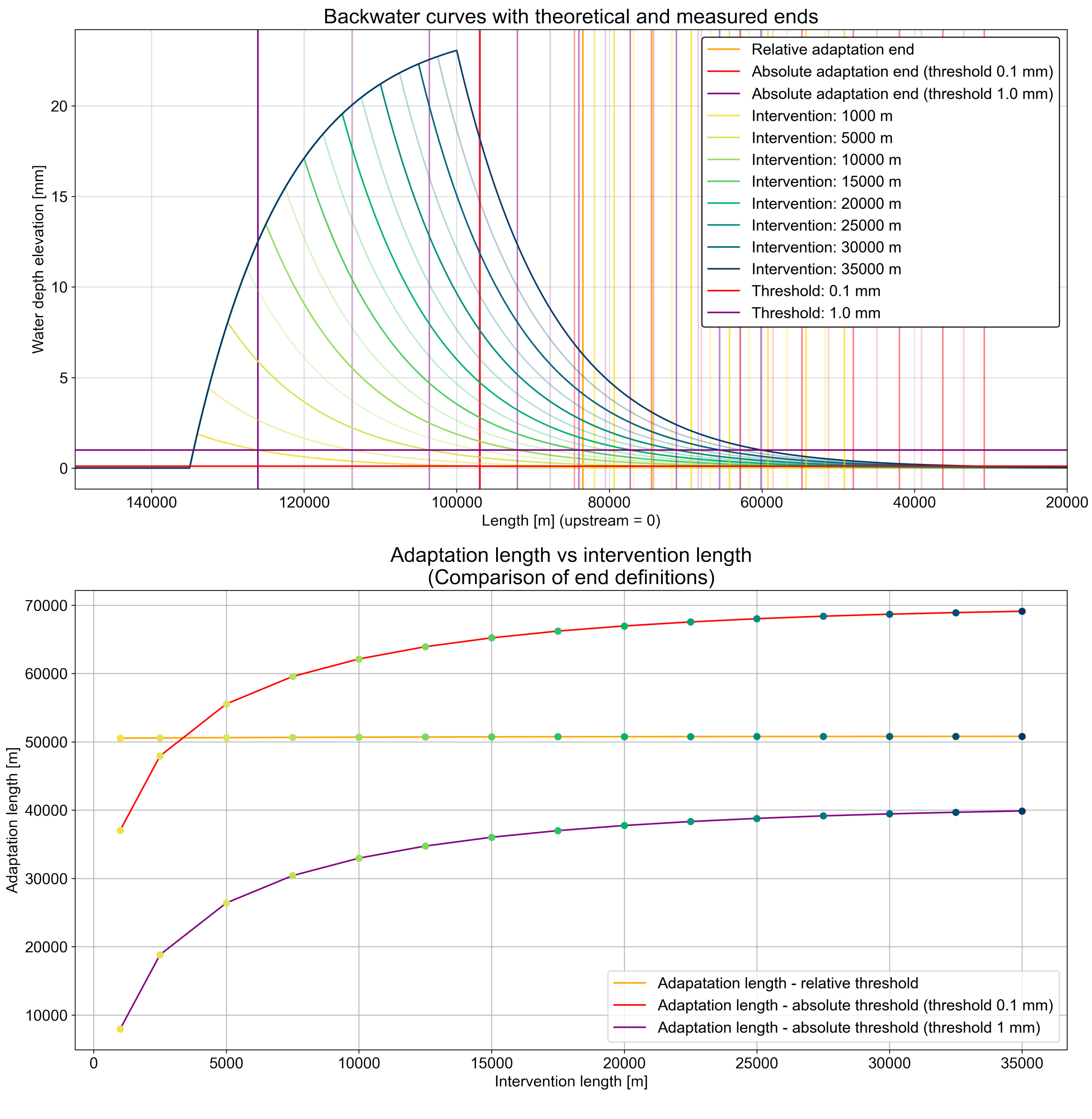

Figure D.1: Comparison between the end point definition of the adaptation length between relative threshold based on Bresse's theory or based on an absolute threshold. Outward expansion of 20 meters is applied with calibrated Waal dimensions.

E

Supporting details for the main dike failure mechanisms

The primary function of a dike is to retain water. A dike is considered to have failed if it no longer fulfils this function or if excessive overtopping occurs. However, dike failure does not necessarily imply a breach. Dike failure can be assessed using a limit state function: $Z = R - S$, where R represents the resistance and S the hydraulic load. Failure occurs when the load exceeds the resistance, i.e., when $Z < 0$ [Kok et al., 2016, Jonkman et al., 2021].
In practice, both the load and resistance are subject to uncertainties. Therefore, safety is often quantified by the probability of failure associated with different failure mechanisms, each contributing to the overall failure probability. This requires full probabilistic modelling, which is computationally intensive and relies on well-calibrated input data. [Kok et al., 2016].
In earlier design stages or feasibility assessments, simpler approaches are often used. These include semi-probabilistic methods or approaches based on the probability of exceedance, and in some cases, fully deterministic methods. These are typically applied to establish preliminary design dimensions or to compare intervention strategies without requiring full reliability analysis [Kok et al., 2016, Jonkman et al., 2021, Slootjes and Van der Most, 2016].

A dike has multiple failure criteria, which could result in losing its water retention function. However, the main criteria associated with dike failure are overflow and overtopping, macro stability inward, macro stability outward, and piping [Van Mierlo et al., 2007, Groenewoud, 2016]. Although other failure criteria also occur, with mainly erosion and micro stability as recurring failure mechanisms of levees, these are fairly dependent on characteristics of the detailed design. Inner slope erosion is also dependent on the criteria used for overtopping. Therefore, these are often not considered during the first failure assessment and design criteria [Groenewoud, 2016], as was the case for the investigated dike reinforcement projects of the Flood Protection Programme (see Appendix C).

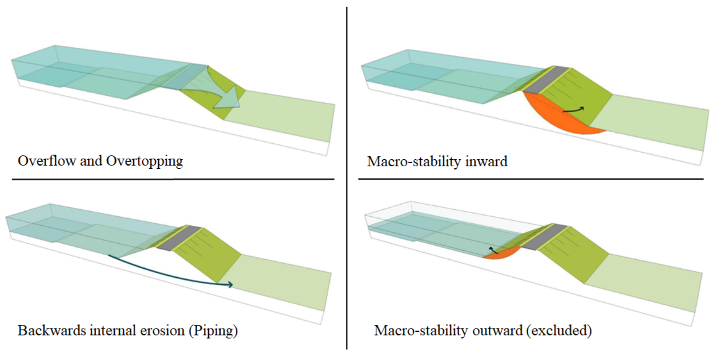

Figure E.1: The four main failure criteria for dikes [Maronier et al., 2018a]

E.1. Overflow and Overtopping

If too much water flows over the crest of the dike, this is deemed a failure. The associated failure mechanisms are overflow and overtopping. Overflow occurs when the still water level exceeds the crest level, allowing water to flow undisturbed over the dike. Any amount of overflow is deemed a failure. Therefore, the dike freeboard height R_c should be higher than the design high water level (DWL). Often, the dike crest height is not solely designed for the overflow criterion but is combined with the overtopping criterion. This is because overtopping can cause failure sooner than overflow alone. Only if the missing (negative) freeboard height R_c is less than 30% of the significant wave height, overflow becomes the dominant failure mechanism [Jonkman et al., 2021].

For the overtopping failure criterion, a limited volume of water per unit of time is permitted to pass over the dike crest. When this threshold is exceeded, the dike no longer fulfils its retention function, which constitutes the overtopping failure mechanism [Van der Meer et al., 2002]. During high water conditions, wave action frequently occurs, causing waves to either overtop the outer crest or run up the slope, reaching higher than the DWL. Depending on the allowable overtopping discharge, a substantial increase in crest height may be required. As a result, the design height for overtopping is added to the crest height determined by the DWL. In practice, overtopping often becomes the dominant and determining factor in establishing the final crest height in dike design, if a surplus height for subsidence and construction measures is disregarded [Jonkman et al., 2021].

Overtopping can be categorised into wave run-up and wave overtopping, both of which are influenced by the geometry of the outer slope and applicable reduction factors. The wave run-up criterion defines the required freeboard height such that only 2% of incoming waves exceed a vertical level relative to the DWL due to run-up. However, this method is relatively conservative and often results in high crest elevations in the absence of reduction measures.
In contrast, the wave overtopping criterion defines the required freeboard height based on a specified mean overtopping discharge per unit time and per unit length of the dike over the outer crest, denoted as q. The freeboard height determined by this criterion is typically lower than that required for wave run-up, although both criteria assume that no failure occurs. The difference arises from how failure is defined in each approach [Van der Meer et al., 2002, Jonkman et al., 2021].

The acceptable amount of overtopping discharge depends on multiple criteria. Firstly, it depends on the stability of the inner slope, while overtopping water can erode the inner side of the dike if the covering layer is not sufficiently resistant. Secondly, the amount of allowable overtopping water is dependent on the usage of the dike and the area behind it. The following applications of the dike determine the amount of critical mean overtopping rates compliant with usage by Rijkswaterstaat and the European Overtopping Manual [Jonkman et al., 2021, Van der Meer et al., 2002]:

- $q \leq 0.1$ l/s per m: considered low or negligible overtopping rates. Applicable for sandy soils with poor grass cover.

  $q < 0.01$ l/s per m: Safe passage for untrained drivers and no damage to buildings.
  $q < 0.1$ l/s per m: Safe passage for aware pedestrians.
- $q \leq 1$ l/s per m: considered to be moderate overtopping rates. Acceptable for clay soil with a proper grass cover.

  $q < 0.5$ l/s per m: Acceptable damage to buildings directly behind the dike.
  $q < 1$ l/s per m: Safe passage for trained staff.
- $q \geq 10$ l/s per m: considered to be high overtopping rates. Careful design for inner slope cover is required. Furthermore, additional drainage assessment is required.

  $q = 10$–50 l/s per m: Access only allowed to trained professionals.
  $q > 10$ l/s per m: Lower bound for the stability of grass-covered inner slope layer exceeded.

The required freeboard height (R_c), is strongly influenced by this permissible mean overtopping rate. For a wave of $H_s = 1$ m, the freeboard height increases by approximately 0.6 m when the mean overtopping rate decreases from $q = 1$ l/s/m to $q = 0.1$ l/s/m [Jonkman et al., 2021]. The amount of overtopping, i.e. the required freeboard height, is furthermore dependent on the slope of the levee (α), the significant wave height (H_s), and the Iribarren number (ξ) [Jonkman et al., 2021], which in turn is dependent on the wavelength (L_0) and thus the wave period ((T_s). The Iribarren number and the equation to convert to the wavelength are shown in Equation E.3.

The wave characteristics are influenced by the fetch length (F), water depth (d), angle of wave attack (β), and wind speed at 10 meters above the water level surface (u_{10}).
The wave height and period are calculated using the empirical Bretschneider wave growth formulas, shown in Equation E.1 [Camarena Calderon et al., 2016], wjle these formulations are implemented in Dutch hydraulic modelling software such as WAQUA and Hydra-NL.

$$\begin{aligned} \bar{H} &= 0.283 \tanh\left(0.530\bar{d}^{0.75}\right) \tanh\left(\frac{0.0125\bar{F}^{0.42}}{\tanh\left(0.53\bar{d}^{0.75}\right)}\right) \qquad \bar{d} = \frac{dg}{u^2}, \quad \bar{F} = \frac{Fg}{u^2} \\ \bar{T} &= 2.4\pi \tanh\left(0.833\bar{d}^{0.375}\right) \tanh\left(\frac{0.077\bar{F}^{0.25}}{\tanh\left(0.833\bar{d}^{0.375}\right)}\right) \qquad \bar{H} = \frac{H_s g}{u^2}, \quad \bar{T} = \frac{T_s g}{u} \end{aligned} \tag{E.1}$$

The 'Technische Adviescommissie voor de Waterkeringen' (Technical Advisory Committee on Flood Defences, TAW) derived a formula to determine the freeboard height based on empirical observations, as shown in Equation E.2, with the right side representing the maximum value [Jonkman et al., 2021]. Although this formula lacks a theoretical foundation compared to the overtopping model by Van der Meer and Bruce (2014), the differences in outcomes are minor for typical or larger design conditions. The TAW formula is commonly applied in assessment software in the Netherlands for dike designs and permitting processes [Jonkman et al., 2021, Van der Meer et al., 2002].

$$\begin{aligned} &\frac{q}{\sqrt{g \cdot H_{m0}^3}} = \frac{0.067}{\sqrt{\tan\alpha}} \cdot \xi_{m-1,0} \cdot \exp\left(-4.75 \cdot \frac{R_c}{\xi_{m-1,0} \cdot H_{m0}}\right) \\ \text{max:} \quad &\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.2 \exp\left(-2.6 \cdot \frac{R_c}{H_{m0}}\right) \end{aligned} \tag{E.2}$$

$$L_0 = \frac{gT^2}{2\pi} \qquad \xi = \frac{\tan(\alpha)}{\sqrt{H/L_0}} \qquad \gamma_b = 1 - 0.0033\,|\beta| \quad (0^\circ < \beta < 80^\circ) \tag{E.3}$$

Additional factors can reduce the required freeboard height needed to prevent wave overtopping failure. These include the presence of berms, surface roughness of the slope, and the obliqueness of incoming waves [Jonkman et al., 2021]. It is assumed that $H_s = H_{m0}$, and that the dike slope is smooth and grass-covered, without berms or additional breaking effects. A damping factor (γ_b) is applied based on the angle of wave attack, with a value of 1 for perpendicular waves and decreasing to a minimum at 80 degrees, as determined with Equation E.3.

The analysis in Section 4.2.1 applies conservative parameter values, or values commonly used in Waal dike reinforcement projects:

- An overtopping discharge of 10 l/s per metre, as typically applied in Waal dike designs [Van Veen et al., 2020].
- An outer dike slope of 1 : 3, frequently used in Waal projects (Appendix C) and considered justifiable from a stability perspective (Subsection 2.1).
- A maximum wind speed of 41.39 m/s [Camarena Calderon et al., 2016].
- Due to data limitations, variations in effective fetch length per wind direction were not incorporated into the analysis.
- To address this limitation, a perpendicular wave attack is assumed, representing the most critical overtopping scenario.
- A maximum fetch length of 15,000 metres, derived using ArcGIS and the baseline schematisation (Subsection 3.1.2), without accounting for wind direction probabilities.

E.1.1. Sensitivity analysis of overtopping parameters under riverward expansion

As outlined in Section 4.1.1, the required freeboard height is influenced by several parameters, including fetch length (F), water depth (d), wave attack angle (β), and wind speed at 10 meters above the water surface (u_{10}). The conclusion that freeboard becomes insufficient for fetch lengths exceeding 3.5 km is based on a highly conservative combination of assumptions: extreme wind speed (41.39 m/s), perpendicular wave attack, long fetch, and peak discharge in the Waal River (see Section 3.1.1). While this represents a worst-case scenario, such conditions are unlikely to coincide in practice.
To illustrate a more realistic but still critical scenario, the main illustrative point ('Hoofdillustratiepunt') was identified using Hydra-NL (see Section 4.1.2). This point reflects the most severe plausible combination of variables for the required freeboard height, for a return period of 1 in 10,000 years at Ochten, the location of which is illustrated in Figure E.2.

Table E.1: Main illustrative point ('Hoofdillustratiepunt') at Ochten with HBN level 13.06 m +NAP and a return period of 10,000 years, computed for the year 2100. The dike orientation is southward, which precludes perpendicular wave attack under these conditions. The values are determined using Hydra-NL with a maximum discharge of 18,000 m^3/s and the W+ climate scenario.

Variable	Value	Uncertainty
Wind direction (contribution to overtopping frequency)	W (21%)	–
Discharge at Lobith [m^3/s]	17955	–
Potential wind speed [m/s]	9.0	–
Local water level [m +NAP]	13.0	0.13
Significant wave height [m]	0.25	0.96
Spectral wave period [s]	1.82	1.03
Wave direction (relative to North) [°]	270.0	–

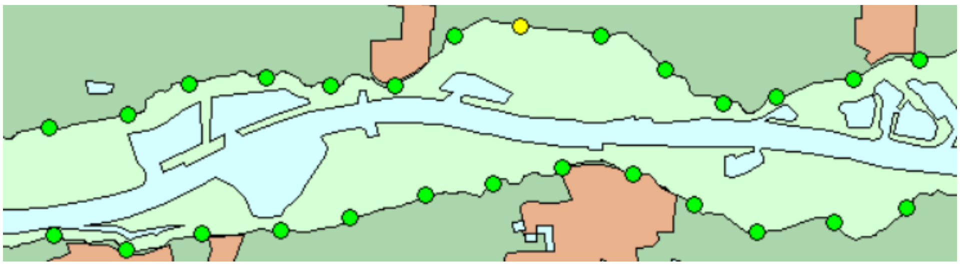

Figure E.2: Visualisation of map in Hydra-NL, with the yellow dot indicating the main illustrative point at Ochten, used for the analysis. The top of the figure indicates North direction.

The angle of wave attack, in combination with the effective fetch length, significantly influences the severity of overtopping, particularly when the wind direction aligns with the longest fetch, resulting in maximum wave generation and direct wave impact. While assuming perpendicular wave attack is a conservative simplification, it does not reflect the directional variability typically observed. This is also evident from the location of the illustrative point, which demonstrates that such extreme alignment does not occur under realistic but critical conditions.

Consequently, the simultaneous occurrence of maximum wind speed, perpendicular wave attack, and extended fetch length, though theoretically possible, is highly improbable. To better understand the individual influence of each parameter on overtopping risk and the required freeboard height, a sensitivity analysis is conducted. This analysis isolates the effects of overtopping discharge, wind speed, dike slope, and wave attack angle, while keeping other variables constant. Table E.2 presents the realistic range of values considered, and the sensitivity of the required freeboard height to each parameter is illustrated in Figure E.3. The analysis with the 1D model is based on 20-metre ODR with the calibrated values of the Waal River, as discussed in Subsection 3.1.1.

Table E.2: Range of influential overtopping parameters used in the sensitivity analysis

Variable	Symbol	Unit	Range	Fixed Value	Source / Basis of Estimate
Overtopping discharge	q	m^3/s per m	0.0001 – 0.01	0.01	[Jonkman et al., 2021]
Wind speed (+10 m surface)	u_{10}	m/s	5.61 – 41.39	41.39	[Camarena Calderon et al., 2016]
Dike slope angle	α	degrees	1:4 – 1:2	1:3	Appendix C
Wave angle of attack	β	degrees	0 – 80	0	[Camarena Calderon et al., 2016]

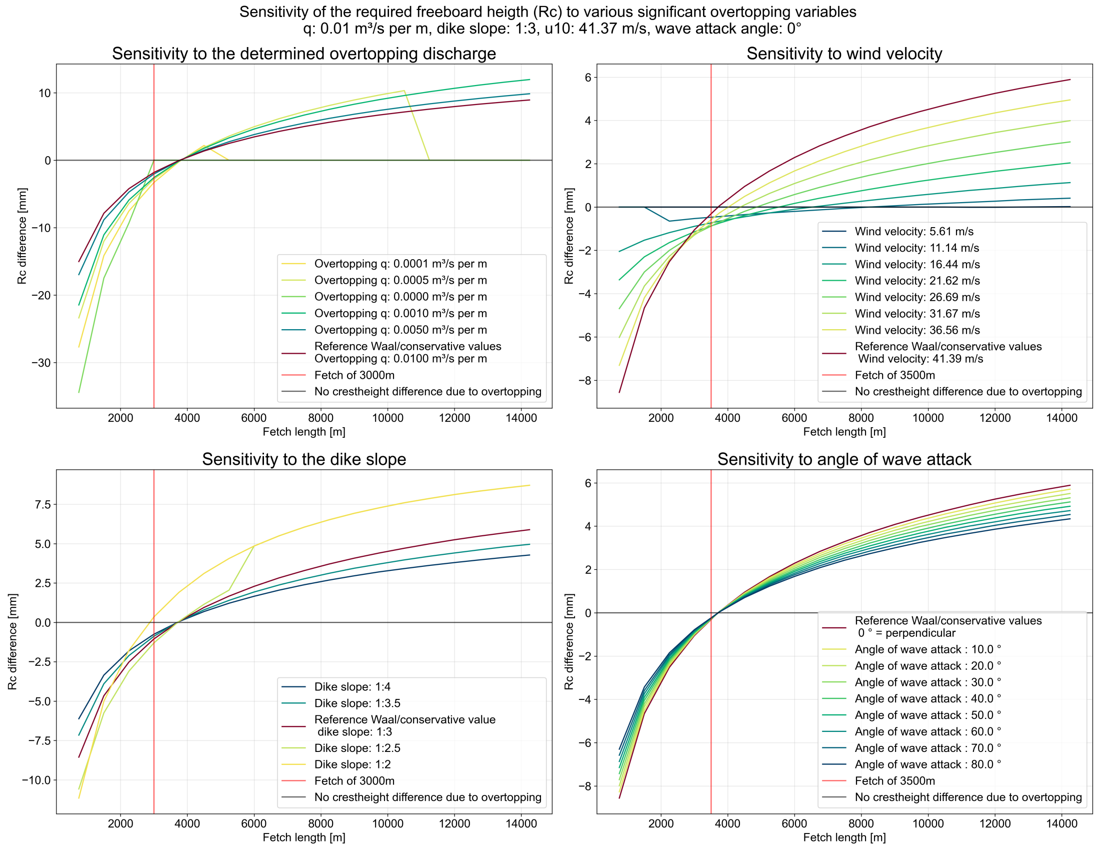

Figure E.3: Sensitivity analysis of influential overtopping parameters: allowed overtopping discharge, wind speed, dike slope, and wave attack angle, on the required freeboard height in response to a water level increase due to outward dike reinforcement. When the influence of a single variable is assessed, all other parameters are held constant at their conservative values.

The effect of outward dike reinforcement on the overtopping mechanism can be considered negligible, as Figure E.3 confirms that fetch lengths below 3.5 kilometres result in negative freeboard height differences under the most realistic scenarios. For plausible combinations, the freeboard height may need to be increased by only a few millimetres, compared to the centimetre-level elevation required for overflow, and is subject to considerable uncertainty. This indicates that overtopping is not a governing failure mechanism under riverward expansion.

Specific sensitivity to influential overtopping parameters affects the overtopping mechanism differently:

- **Overtopping discharge:** Stricter criteria increase the required crest height beyond a 3.5-kilometre fetch. In extreme cases, the criterion becomes so conservative that a 2.3 cm water level increase no longer affects the required freeboard height.
- **Wind speed:** Wind speed has the most pronounced effect on freeboard height. Higher wind speeds significantly increase freeboard requirements, especially with longer fetches. Yet, as shown in Table E.1, combinations of high wind speed and optimal wind direction for maximum overtopping are highly unlikely.
- **Dike slope:** Steeper slopes reduce overtopping sensitivity, but slopes steeper than 1 : 3 are not permitted (Section 2.1). Shallower slopes are more vulnerable to water level increases.
- **Wave angle:** Non-perpendicular wave attack reduces the required freeboard.

E.2. Stability

Macro-instability is the failure mechanism that occurs when a large soil mass of the dike body, larger than approximately one meter, shears and slips, causing the dike to lose its primary function of water retention, as shown in the right two cross-sections in Figure E.1. This mechanism can develop on either the inner or outer slope, depending on the specific conditions ['t Hart et al., 2016].

A stable dike is in equilibrium. The weight of the dike slope generates gravitational forces along potential slip planes, which contribute to both the driving and resisting moments. The hinterland behind the slope provides additional passive resistance, contributing to overall slope stability. For the slope to remain stable, the driving moment must not exceed the resisting moment provided by the mobilised shear strength of the soil, thus maintaining equilibrium [Knappett and Craig, 2012]. Otherwise, instability due to shearing occurs and the slope slips along a circular or straight path ['t Hart et al., 2016]. The driving and resisting moments of a slope are influenced by various factors, including soil properties, slope geometry, and the presence of water. Taylor's chart, which is based on the limit equilibrium method (LEM) with circular slip surfaces, remains one of the most widely used approaches due to its simplicity and near-accurate results [Sahoo and Shukla, 2019]. According to this method, steeper slopes tend to develop toe failure circles, which are generally associated with lower factors of safety (ratio between resisting and driving moments). Therefore, in the case of undrained slopes below the water table, it can be concluded that steeper slopes are typically less stable [Steward et al., 2011]. However, dike slopes are often subject to water and infiltration during floods or high water levels occurs, which increases pore pressures, reduces effective stress, and lowers the shear strength capacity. This can ultimately lead to loss of equilibrium ['t Hart et al., 2016]. As such, pore water conditions are a critical consideration in dike stability analysis [Jonkman et al., 2021].

High river levels can raise the phreatic surface within a dike, increasing pore pressure and reducing the mobilised shear strength of the soil in the dike. For primary flood defences in The Netherlands, all soil is assumed to be undrained, so that pore pressure can not slowly dissipate and thus increase the effective stress. The rate at which the phreatic surface responds to rising water levels depends on the soil permeability ['t Hart et al., 2016]. The maximum height the phreatic surface can reach in the dike depends on the duration of high water levels and the height itself. For dike design, the worst-case scenario (i.e., highest phreatic surface) is used during stability analysis [Jonkman et al., 2021]. Beside the dike body, also the foundation and the toe of the dike might become saturated, reducing the resisting moment. The aquifer layer beneath the foundation, which can extend far inland, is often more permeable than the foundation layer. During high water, the rise in pore water pressure occurs faster in the aquifer. This imbalance of pore water pressure between the foundation and aquifer might result in the aquifer pushing the foundation layer up if the dike and the foundation layer do not have sufficient self weight, which is called Uplift ['t Hart et al., 2016]. Due to the pore water pressure in the aquifer, the effective stress at the boundary of the foundation layer drops, causing the foundation layer to lose its shear resistance and resulting in a loss of soil equilibrium. This can lead to either horizontal sliding of the entire dike due to water pressure from the river or deformation of the foundation layer and consequently the dike. Therefore, it is crucial that the dike and its foundation inhibit sufficient self weight [Jonkman et al., 2021, 't Hart et al., 2016].

If the resisting forces become smaller than the driving forces due to the increase in pore pressure, instability may occur, forming a slip surface. Following this slope failure, a new equilibrium may form, and the dike might still retain its water-holding function if the slip surface does not compromise the outer slope, but only the crest or inner slope. However, other mechanisms, such as erosion or hydraulic pressure, may still lead to a breach. Cracks often appear before full failure, and the time between their appearance and collapse depends on soil properties ['t Hart et al., 2016].

Determining the stability of a slope is often done with either Limit Equilibrium Method (LEM) like Taylor's Chart or Bishop, or numerical Finite Element Method models (FEM). LEM are used for their simplicity over numerical models, but this is combined with many assumptions that limit their use. So is it not possible to integrate soil behaviour combined with a groundwater flow analysis, saturated slopes or to integrate structures in the assessment. While in the past calculations were done by hand, this is commonly done with simplified modelling software like D-Geo Stability. FEM is more complex, while FEM is able to account for these soil and groundwater measures, but are complex and time consuming and require many variables, which are often not instantly available. [Jonkman et al., 2021].

As a rule of thumb for inner macro stability during high water, an inner slope with an average angle shallower than 1:4 is generally not problematic, if berms are included [Jonkman et al., 2021]. The steeper the slope, the higher the chance of instability ['t Hart et al., 2016, Steward et al., 2011]. However, the self-weight of the soil must still be sufficient to counteract uplift. Steeper slopes are possible if additional measures are taken. After detailed design, a slope of 1:3 is justifiable, which is often used in practice, as seen in Section C. These measures include adding an impermeable layer on the outer dike slope and toe to reduce water infiltration and the rise of the phreatic surface. Additionally, a drainage basin behind the dike can lower the phreatic surface. If berms, impermeable layers, or sufficient space for self-weight retention are not possible, sheet piles or concrete retaining walls can be used for soil retention, either in the crest or the toe. In the toe, the soil-retaining function is maximised, but these are harder to implement, while there is often not sufficient working space and these have to be applied from the crest (C. Spoorenberg, personal communication, May 2025). If sheet piles are implemented in the crest, sheet piles can mitigate the effect of water infiltration [Jonkman et al., 2021].

For the outer slope macro stability, the same principles apply as for the inner slope. However, during high water, the outer slope is not the critical failure mechanism. The water from a flood pushing against the outer slope acts as an additional resisting moment, maintaining outer slope equilibrium ['t Hart et al., 2016]. Only if the river water level drops too quickly for the phreatic surface to follow, resulting in low shear strength in the outer slope without the water's resisting moment, might the dike fail. This is called a 'sudden drawdown' [Jonkman et al., 2021]. However, since the high water level has receded, this does not result in immediate inundation and loss of the water retention function ['t Hart et al., 2016].

E.3. Backward internal erosion

When there is a difference in hydraulic head between the river side and the inner side of the dike, water may start to flow underneath the dike's foundation layer. This flow can erode soil particles, gradually weakening the structure. Eventually, this process may lead to a complete loss of soil stability, causing the dike to collapse. This failure mechanism is known as backward internal erosion, or more commonly, piping. Piping consists of multiple sub mechanism, before the dike completely collapses ['t Hart et al., 2016].

First, due to a difference in hydraulic head between the river water level and the phreatic surface behind the inner side of the dike, the pore pressure of the aquifer increases, leading to uplift as discussed earlier in Appendix E.2. This uplift can create ruptures in the soil behind the dike, especially where the depth between the aquifer and the mean surface is thin and poses low resistance. Groundwater may then seep under the foundation layer to these ruptures. If the hydraulic head gradient between the river and the exit point at the rupture exceeds a critical value, soil particles start to erode under the poorly permeable aquitard layer beneath the foundation layer (or the foundation layer itself), starting from the exit point. This is called Heave. This process initiates the formation of a backwards erosion pipe, a small channel of water formed by eroded particles which uses the aquitard or poorly impermeable foundation layer as a roof, which propagates towards the river. If this pipe formation is not halted due to soil conditions or collapses, it will accelerate due to the increased loss of hydraulic resistance. Eventually, contact with the river is made, causing the entire dike's foundation layer to lose its shear capacity, leading to failure.

From a system reliability perspective, backward internal erosion is modelled as a parallel failure system, in which the overall failure mode only occurs if the limit states of uplift, heave, and piping are all exceeded simultaneously. Therefore, mitigation measures targeting any one of the sub-mechanisms can increase the overall safety against piping failure [Jonkman et al., 2021, 't Hart et al., 2016].

In the Netherlands, piping is assessed based on the critical hydrauic head difference or piping gradient. To asses the piping criteria, Sellmeijer's equation is used, as shown in E.4. The equation of Sellmeijer is chosen over Bligh and Lane's equation, while these formulas underestimate the required seepage length [Jonkman et al., 2021]. The meaning of each symbol can be found in Table E.3.

$$Z_p = H_c - (h - h_p - 0.3d) \qquad \text{(E.4)} \qquad\qquad H_{c,p} = F_1 F_2 F_3 L \qquad \text{(E.5)}$$

$$F_1 = \eta \left(\frac{\gamma_s}{\gamma_w} - 1 \right) \tan\theta \quad \text{(E.6a)} \qquad F_2 = \frac{d_{70m}}{\sqrt[3]{\frac{\nu k L}{g}}} \left(\frac{d_{70}}{d_{70m}} \right)^{0.4} \quad \text{(E.6b)} \qquad F_3 = 0.91 (D/L)^{\frac{0.28}{(D/L)^{2.8}-1}} + 0.04 \quad \text{(E.6c)}$$

While Sellmeijer's assessment is dependent on the piping gradient ($i = Hc/L$), the seepage length is a key factor in avoiding the failure mechanism. Therefore, often the seepage length dictates the length and dimensions of a dike. The longer the seepage length, the smaller the piping gradient. Often, a berm is chosen to prolong the seepage length. The rupture is pushed further inland of the dike, while also the hydraulic gradient is reduced, because the additional berm will be saturated with groundwater. Lastly, a berm has additional weight that counteracts uplift and contributes to the resisting moment for stability. The seepage length at the riverside can be extended by applying an additional impermeable cover before the cover layer or aquifer starts. Vice versa, if the cover layer at the outer side of the dike is eroded or excavated, it poses additional risks for piping. If there is insufficient room for berms to extend the seepage length, sheet piles can be used as a mitigation measure. They can either extend the seepage length, requiring the erosion channel to overcome the vertical sheet pile depth twice, or completely cut off groundwater flow from the aquifer. The hydraulic head difference could also be reduced. This could be done by creating a small water catchment behind the dike or by elevating the water level in the ditches behind the dike. that results in a lower head difference. Lastly, the water pressure could be released by drainage systems with filters that allow flow but block soil particles [Jonkman et al., 2021].

The ultimate limit state of heave is defined as the situation where the slope over the height of the seepage screen is equal to the slope fluid at which fluidisation occurs in the grain skeleton [Spierenburg and Calle, 1998]. Also, to ensure sufficient safety against heave, in the case of a vertical seepage flow in sandy soil behind a seepage screen, the maximum slope that occurs must be smaller than the slope at which heave occurs [Förster et al., 2012]. Therefore, if the heave criterion is not met, Sheet piles are used to increase the hydraulic upward gradient by increasing the vertical seepage length. For the heave limit state, the formula by Sellmeijer is applied, as shown in Equations E.7 till E.10, while this is commonly applied in practice (C. Spoorenberg, personal communication, May 2025).

$$Z_h = i_{c,h} - i \qquad i_{c,h} = \frac{(1-n)(\gamma_s - \gamma_w)}{\gamma_w} \approx 1.65(1-n) \qquad i = \frac{\varphi_{\text{exit}} - h_p}{d} \tag{E.7}$$

$$\varphi_{\text{exit}} = h_p + \lambda (h - h_p) \tag{E.8}$$

$$\lambda = \frac{\lambda_h}{L_f + B + \lambda_h} \exp\left(\frac{B/2 - x_{\text{exit}}}{\lambda_h} \right), \quad x_{\text{exit}} > \frac{B}{2} \tag{E.9}$$

$$\lambda_h = \sqrt{\frac{kDd}{k_h}} \tag{E.10}$$

The limit state for uplift is given by Equation E.11, where d represents the thickness of the blanket layer. It can be concluded that this thickness and the soil parameters mostly determine the uplift criterion [Jonkman et al., 2021].

$$Z_u = \Delta\varphi_{c,u} - \Delta\varphi \qquad \Delta\phi_{c,u} = d \cdot \frac{\gamma_{\text{sat}} - \gamma_w}{\gamma_w} \qquad \Delta\varphi = \varphi_{\text{exit}} - h_p \tag{E.11}$$

E.3.1. Determination and sensitivity of required sheet pile lengths

This section outlines the method used to determine the required sheet pile length for the four conceptual dike designs at Ochten, followed by a sensitivity analysis of key soil parameters. The analysis focuses on internal backward erosion, which consists of three sub-mechanisms: piping, heave, and uplift.

In practice, the assessment of internal erosion begins with evaluating piping resistance. If this is insufficient, the heave criterion is assessed. When both criteria indicate failure, reinforcement is required. The required sheet pile length is governed by the sub-mechanism that demands the shortest seepage length. This is typically governed by the heave criterion, because vertical transport of soil particles, driven by the phreatic head, is considered significantly more difficult than horizontal transport (C. Spoorenberg, personal communication, May 2025). Therefore, a more in-depth approach for the heave calculation is discussed.

The variables that dictate the heave criterion are site-specific and could vary spatially within short ranges. To maintain clarity, this study evaluates a single representative subsoil profile, based on the reference cross-section near Ochten, supplemented by literature and data from an anonymised dike reinforcement project near the Waal (C. Spoorenberg, personal communication, May 2025). The parameters used are listed in Table E.3. Seepage lengths are derived from the geometry of each conceptual design (Section 2.2) and conservatively measured from the inner to the outer toe of the dike.

According to the system reliability representation of backward internal erosion, failure occurs only if the limit states of uplift, heave, and piping are simultaneously exceeded. However, uplift is not included in the present analysis. This exclusion is based on the observation that the uplift limit state is not exceeded under any realistic scenario with the variables presented in Table E.3. By omitting uplift, the effects of water level differences on the sub-mechanisms of heave and backward erosion (Sellmeijer) can be meaningfully assessed. This allows for evaluating the impact of outward dike reinforcement on these mechanisms, which is relevant for cost estimation and hydraulic design considerations.

Several methods exist to determine this critical heave gradient. In this study, Terzaghi's formulation is applied [Jonkman et al., 2021], which depends on the porosity of the aquifer (Equation E.7).
To determine the gradient of the actual water head, the potential piezometric head in the aquifer at the assumed exit point (φ_{exit}) is required, which is shown in Equation E.8. This exit point is conservatively taken at the inner toe of the dike unless specific hinterland seepage lengths are investigated. The potential head depends on the phreatic levels at the riverside and hinterland, as well as subsurface properties.
The potential head at the exit point is further influenced by the damping factor λ, based on horizontal flow with vertical leakage assumption, which depends on the hinterland leakage factor λ_h. The potential head at the exit point, using the damping factor, is in practice commonly assessed with a derivation by Sellmeijer of the damping factors (C. Spoorenberg, personal contact, May 2025), as shown in Equation E.10 [Jonkman et al., 2021].

The damping factor formula by Sellmeijer is just one possible analytical derivation, but this can vary based on assumptions and groundwater flow models [Jonkman et al., 2021]. It is dependent on the assumed location of the exit point x_{exit}, which should be larger than half of the dike width measured from the axis of the dike crest $B/2$. In this study, this is set to the inner toe, except when the effect of the hinterland seepage length is investigated. It is furthermore influenced by the foreshore seepage length L_f.

Although the reference cross-sections at Ochten are designed for an HBN of 13.44 m+NAP, applying a slightly lower DWL of 12.5 m +NAP avoids the unrealistic outcome of all conceptual designs requiring sheet piles, which is not the case for the observed situation. This discrepancy likely results from longer actual seepage lengths in the reference profiles, which cannot be reliably estimated due to a lack of subsurface data. Therefore, a DWL of 12.5 m +NAP is adopted as the phreatic river level, as it best approximates the hydraulic conditions of the reference cross-sections. This choice does not affect the sensitivity analysis, which is governed by head differences, while changes in either the phreatic exit level or DWL yield the same sheet pile length.

If sheet piles are required to increase the hydraulic upward gradient for heave, the thickness of the blanket layer is automatically added to the sheet pile length. Although this layer thickness is already accounted for in the heave calculation, it must be penetrated by the sheet pile to contribute to the hydraulic gradient. Furthermore, sheet pile lengths are rounded up to half-metre increments, in line with common practice by contractors in dike reinforcement projects (P. van der Scheer, personal communication, April 2025).

Table E.3: Geotechnical and hydraulic variables for piping analysis based on Waal conditions

Variable	Symbol	Unit	Range	Fixed Value	Source / Basis of Estimate
Volumetric weight of sand grains	γ_s	kN/m^3	–	26.5	[Jonkman et al., 2021]
Volumetric weight of water	γ_w	kN/m^3	–	10	[Jonkman et al., 2021]
Bedding angle	θ	degrees	–	37	[Van Esch, 2014]
Drag factor coefficient	η	–	–	0.25	[Van Esch, 2014]
Kinematic viscosity of water	ν	m^2/s	–	1.33×10^{-6}	[Jonkman et al., 2021]
70% fractile of the grain size distribution	d_{70}	m	–	3.5×10^{-4}	Anonymised Waal project
Reference value for d_{70}	d_{70m}	m	–	2.08×10^{-4}	[Jonkman et al., 2021]
Gravity constant	g	m/s^2	–	9.81	–
Floodplain level	–	m+NAP	–	7.00	AHN viewer cross-section Ochten
Phreatic exit level Hinterland level	ϕ_{exit}	m+NAP	4.00 – 7.79	7.786	Cross-section at Ochten
Thickness of top layer	–	m	–	0.786	Anonymised Waal project
Thickness of aquitard (Holocene sand)	d	m	0.3 – 2.5	1.336	Anonymised Waal project
Thickness of blanket layer	d	m	1.09 - 3.3	2.122	Combination of top layer and aquitard
Hydraulic conductivity of aquitard	k_h	m/s	–	1.736×10^{-4}	Anonymised Waal project
Thickness of aquifer	D	m	0 – 40	19.25	Anonymised Waal project
Hydraulic conductivity of aquifer	k	m/s	–	9.735×10^{-4}	Anonymised Waal project
Additional hinterland seepage length	L_h	m	–	0	Conservative estimate
Additional foreshore seepage length	L_f	m	–	0	Conservative estimate
Seepage length Inward reinforcement	–	m	–	55.4	Section 2.2
Seepage length Construction-based reinforcement	–	m	–	44.4	Section 2.2
Seepage length Outward 20-metre reinforcement	–	m	–	62.9	Section 2.2
Seepage length Tuimeldijk	–	m	–	52.1	Section 2.2

Sheet pile calculation and sensitivity of variables

The required sheet pile lengths for heave and piping are calculated using the parameters listed in Table E.3. Initially, the limit states are assessed without applying any sheet piles. If one of the criteria is not satisfied, sheet pile length is incrementally increased until the relevant limit state is just met. As previously discussed, this always turns out to be the heave criterion. The resulting exceedance-based limit states for each conceptual design are summarized in Table E.4.

Table E.4: Overview of piping and heave limit states for the four conceptual designs. For each variant, the limit states are first evaluated without sheet piles. If the criteria are not met, sheet piles are added incrementally until the most efficient (minimal) length is found that satisfies either the piping or heave criterion.

	Inward reinforcement	Construction-based	Outward reinforcement	Tuimeldijk
Z-limit state piping	-1.33	-1.81	-1.01	-1.48
Z-limit state heave	0.025	-0.062	0.071	0.002
Exact Sheet pile length [m]	0.00	2.39 (Z heave: 0.001)	0.00	0.00
Rounded Sheet pile length [m]	0.0	2.5 Z heave: 0.022)	0.0	0.0

Additionally, a sensitivity analysis is performed to evaluate the impact of variations in water head difference (via changes in DWL or phreatic exit level), blanket layer thickness, and aquifer thickness. This analysis illustrates how weaker soil conditions influence the internal backward erosion mechanism. Soil parameters are kept constant, acknowledging that a wide range of combinations may apply to different soil types.

Figure E.4 shows that, for the given parameters, the heave criterion governs the required sheet pile length. Only the construction-based design requires sheet piles; however, the pre-installed 5-metre sheet piles for stability are already sufficient. The other three designs do not require additional sheet piles, although the Tuimeldijk design appears to be marginally sufficient against internal backward erosion.

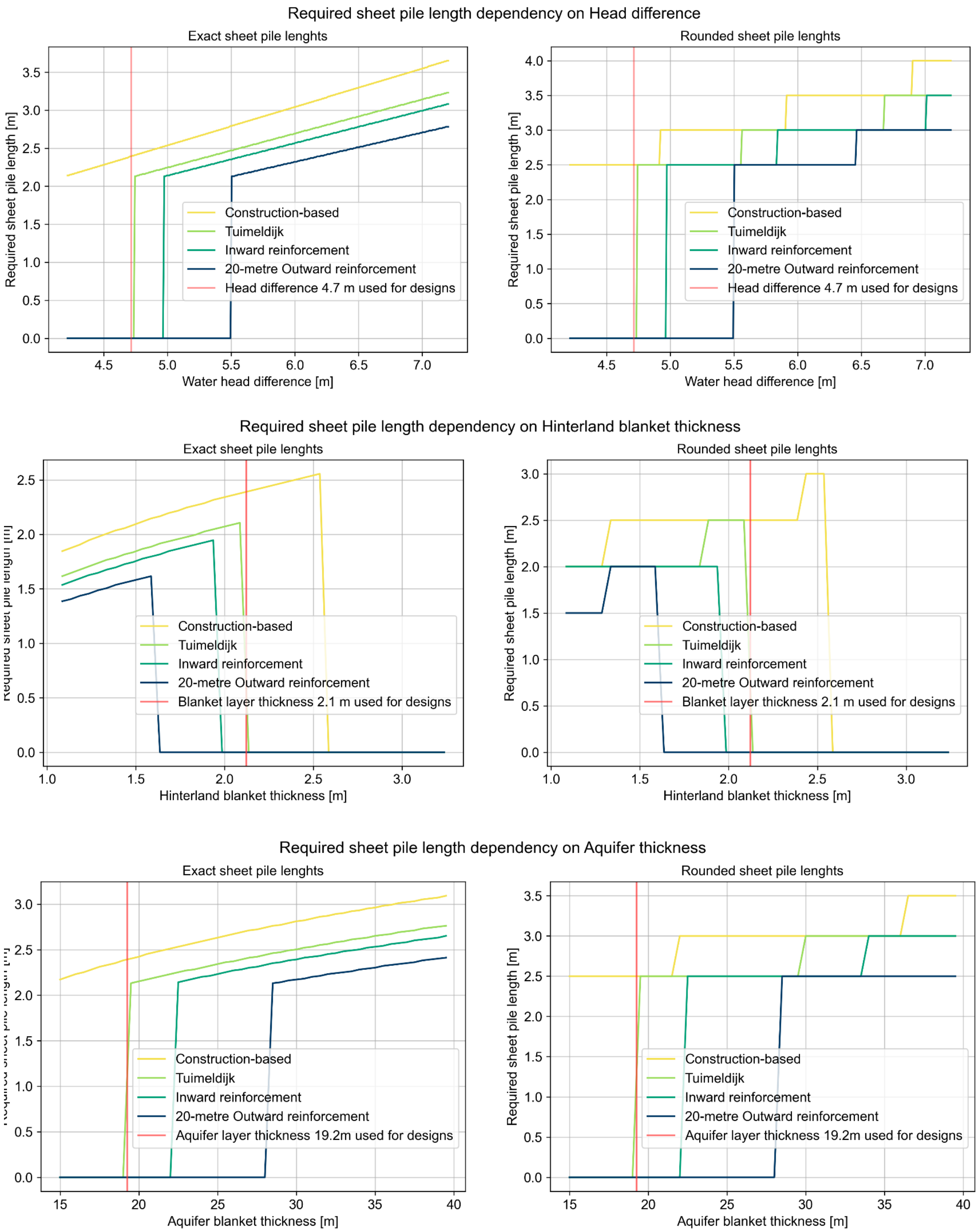

Figure E.4: Sheet pile length calculation for the four conceptual designs and a sensitivity analysis on water head difference, hinterland blanket thickness and aquifer thickness.

F

Schematisation of outward dike reinforcement in D-Hydro

In this appendix, the modifications made to the Baseline 7 model schematisation provided by Rijkswaterstaat are described, to clarify how these adjustments influence the D-Hydro simulations.

Used schematisation

The original schematisation used is baseline-rijn-beno19_6-v2, which dates from 2019 and does not include more recent updates to the northern riverbanks, as discussed in Appendix C. While this omission does not affect the analysis of outward dike expansion, it may influence the model calibration described in Appendix G. Therefore, the measures and variants presented here should not be interpreted as absolute representations of current conditions. This version was provided by Rijkswaterstaat, as a more recent and complex schematisation was not deemed necessary for the exploratory nature of this study (Personal communication, Rijkswaterstaat, July 2025).

Schematisation of Outward dike reinforcement

To simulate the effects of outward dike expansion in the Baseline schematisation and D-Hydro simulations, flow_blocking_polygons were added along the riverside of the elevated_line_routes. These polygons represent the flow-blocking effect of the dike crest. However, it should be noted that the elevated_line_routes in the Baseline schematisation do not always correspond precisely to the actual dike alignment. In some locations, the line follows the dike crest axis, while in others it aligns with the outer edge of the crest (Personal communication, Rijkswaterstaat, July 2025). Although this does not fully reflect the real-world situation along the Waal, the flow_blocking_polygons were consistently applied relative to the elevated_line_routes. As a result, this inaccuracy is considered negligible when comparing variants to the baseline.

Each modification to the primary dike system was implemented by creating a copy of the baseline-rijn-beno19_6-v2 schematisation. The intended changes were applied in a measure file, which was then converted into a variant. This variant was subsequently transformed into a valid D-Flow FM schematisation, in accordance with the Basisrapport WBI 2017 [De Waal, 2018, Spruyt et al., 2024], for use in design exploration and hydraulic analysis.

The outward dike expansion was schematised by generating flow_blocking_polygons of varying widths, referenced to the elevated_line_routes. This was done using the Buffer tool in ArcGIS Pro, as illustrated in Figure F.1. To integrate the measure into the D-Hydro suite, several additional steps were taken: overlapping polygons were removed, a measure contour polygon was created, and only the newly added flow_blocking_polygons were assimilated into the variant. The variant was then converted into a D-Flow FM schematisation using the Baseline 7 tool. Finally, only the newly generated *_dry.pol, *_enc.pol, and *_thd.pli files were updated in the Rijn.mdu file to enable D-Hydro simulations.

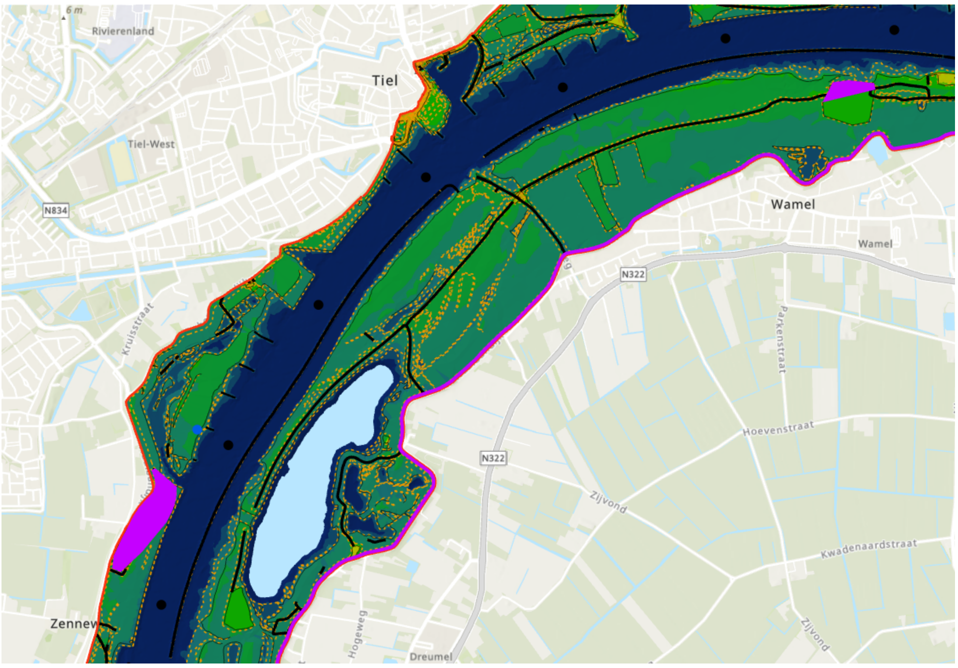

Figure F.1: Visualisation of applying flow-blocking polygons via the Buffer tool in the Baseline schematisation

Created variants

Using the described method for simulating outward dike expansion, a total of eleven variants were developed to analyse its hydraulic effects.

The first four variants ($a1$ to $a4$) apply outward expansion along the entire river reach, each with a different expansion width. These were designed to calibrate the 1D model to accurately reflect the impact of varying expansion magnitudes. In this context, the 'entire river reach' refers to the section of the Waal between Dreumel (river kilometre WL_921) and Nijmegen (WL_887). This specific reach was selected to ensure that tidal dynamics and discharge distribution effects from the Pannerdensche Kop do not influence the observed water level differences during the intervention.

Subsequently, a fixed outward expansion of 20 metres was applied in variants $a5$ to $a9$, each with a different intervention length. The start and end river kilometres, as well as the corresponding intervention lengths, are summarised in Table G.1.

Notably, variant $a8$ is the only case where the intervention does not start at WL_921, but instead around WL_912. This was done to investigate the influence of a wider river section compared to the narrower reach near WL_921. As this variant was not used for calibration (Appendix G), it serves as a valuable case for validating the simplified 1D model. With an intervention length of 10 km, a very roughly estimated average floodplain width of 1.5 km, and a measured water level difference of 1 cm, the 1D-model slightly over-predicts the response by a few millimetres.

Lastly, variants $a10$ and $a11$ explore outward reinforcement, either excluding or specifically targeting bottleneck locations. These bottlenecks were identified based on the results of other variants. Subsequently, a dike segment was classified as part of a bottleneck if located within a 300 – 400 m radius of the river edge, typically where groynes are present, and if its position aligned logically with the flow trajectory.

Variant $a10$ applies outward expansion along the entire dike except at bottlenecks, while variant $a11$ applies reinforcement solely at the bottlenecks. The schematisation of these two variants are shown in Figures F.2 and F.3 respectively.

Because the variants were not used in calibration, the cases provide an out-of-sample basis for validation. As shown in Figure 3.10 in Section 3.2.2, the bottleneck at Beneden-Leeuwen is associated with a water-level increase of about 5 mm. The corresponding intervention length is approximately 2.5 km, with an estimated average floodplain width of around 1,000 m.

At Beuningen, a similar rise of 3 – 4 mm is indicated. The associated intervention length is approximately

1.5 km, with a floodplain width of around 500 m. These dimensions were approximated from the baseline schematisation in ArcGIS.
The resulting water-level responses are broadly consistent with the simplified 1D approximations (Figure 3.15). This consistency supports the use of the approximations for smaller, segmented outward-expansion scenarios.

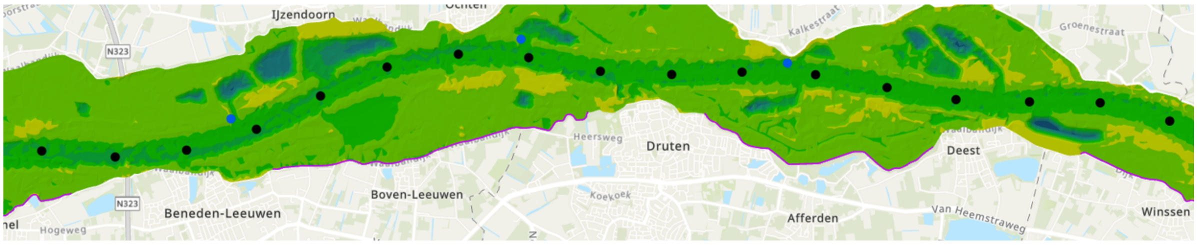

Figure F.2: Variant $a10$: Only outward reinforcement outside bottlenecks

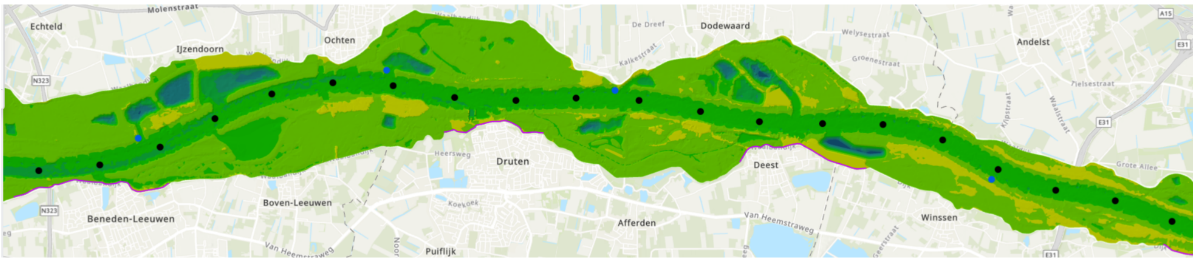

Figure F.3: Variant $a11$: Only outward reinforcement in Bottlenecks

Table G.1 shows an overview of all the applied variants in this study.

Table F.1: Model schematisation variants for outward dike expansion analysis

Variant	**Start intervention**	**Intervention length (est.) [km]**	**Outward expansion amount [m]**
a1	WL_921.0	34	20
a2	WL_921.0	34	5
a3	WL_921.0	34	10
a4	WL_921.0	34	15
a5	WL_921.0	10	20
a6	WL_921.0	17	20
a7	WL_921.0	22	20
a8	WL_912.2	10	20
a9	WL_921.0	5	20
a10	-	No bottlenecks	20
a11	-	Only bottlenecks	20

G

1D model calibration based on D-Hydro suite simulations

This appendix describes the preprocessing of simulation outputs from the D-Hydro Suite and the subsequent calibration of the 1D hydraulic model, as introduced in Section 3.1.1.

Model Setup

Water level simulations were conducted using the D-Hydro Suite FM model version 2023.02. The baseline runs are based on the Baseline 7 and D-Hydro schematisation of the Rijn and Waal rivers provided by Rijkswaterstaat: baseline-rijn-beno19_6-v2 and dflowfm2d-rijn-beno19_6_20m_waal-v2d. This model configuration was granted for use by Rijkswaterstaat. Although version 2025.01 of the D-Hydro Suite was available, it could not be executed on the designated workstation and returned only initial conditions, rendering it unsuitable for this study.

Cleaning and detrending of the D-Hydro simulations

A full range of discharge simulations, excluding low-flow conditions, was performed to serve as the baseline for calibrating the 1D model. No modifications were made to the original schematisation provided by Rijkswaterstaat during these simulations. The resulting HIS.output files were converted to Excel format and filtered to retain only water level data between river kilometres WL_925 and RH_848. Data upstream of this range were unavailable, while downstream sections are influenced by tidal dynamics and backwater effects from the sea. To maintain analytical clarity, these regions were excluded. The selected stretch covers the area from Dreumel, past the Pannerdensche Kop, up to Lobith.

To ensure data quality, duplicates and irregular entries, such as those associated with divers and weirs, were removed, leaving only clean river kilometre data, derived from the middle axis of the rivers. The cleaned datasets for the various baseline simulations are visualized in Figure G.1.
Since the 1D model is based on water depth relative to the riverbed, the natural slope of the river was removed from the water level data. The slope correction was based on the lowest discharge simulation ('S_3000'), using linear regression to estimate an average slope of $1.01\ 10^{-4}$. This value slightly deviates from the slope reported in literature for the Waal, which is approximately $1.05\ 10^{-4}$ [Domhof et al., 2018]. The detrending was performed relative to river kilometre WL_901, where the bed level was determined to be approximately +NAP = 0 using the Baseline schematisation provided by Rijkswaterstaat. Upstream of this point, the slope was subtracted, downstream it was added. This approach allows the water level data to be referenced back to NAP if needed. The detrended water levels from the baseline simulations are also shown in Figure G.1.

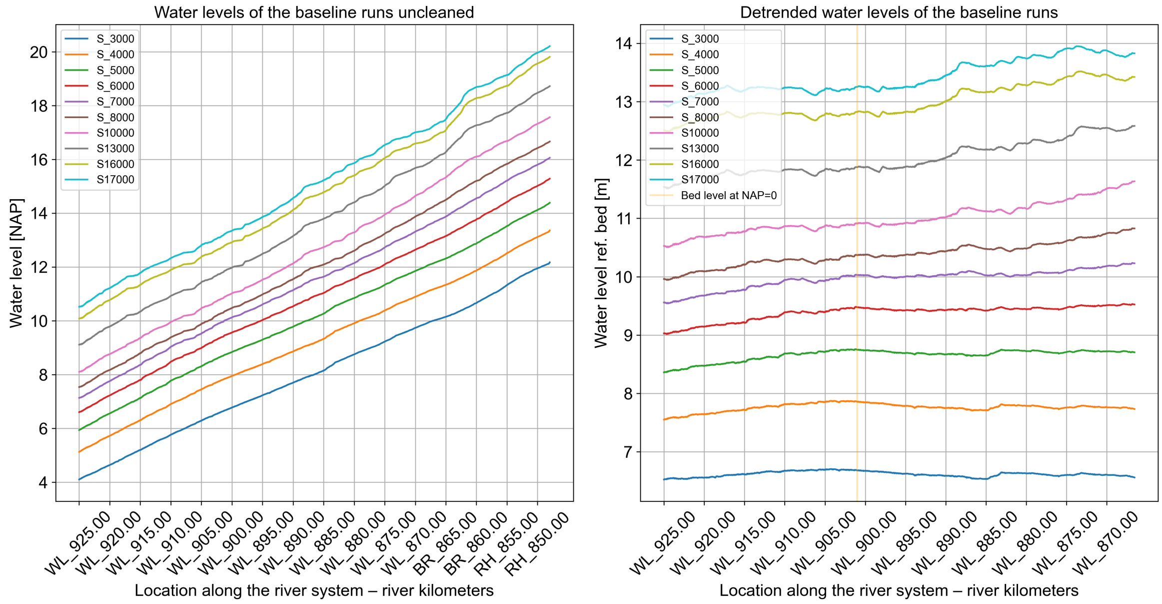

Figure G.1: Cleaned and detrended water levels from the baseline D-Hydro Simulations

1D simplified compound channel model calibration

From the detrended baseline simulations, the average water level relative to the bed level was extracted. This yielded a direct relationship between water level and discharge, derived from the D-Hydro outputs. This relationship is illustrated in Figure G.2, and serves as a reference for comparison with the 1D model results, forming one half of the calibration process. Although interpolation was applied to smooth the data, it was deliberately excluded from the calibration procedure to avoid introducing a false sense of model reliability or overconfidence in the results.

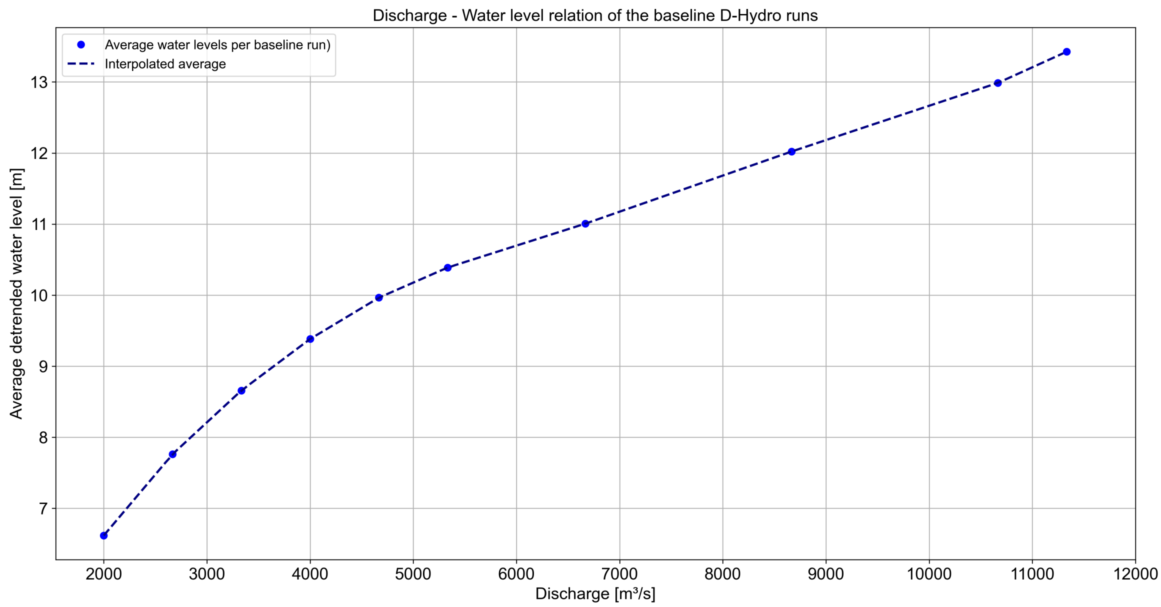

Figure G.2: Water level–discharge relationship based on D-Hydro baseline simulation

In addition to the reference simulations, the effect of outward dike expansion was also investigated. The modifications made to the baseline schematisation to simulate these expansions are described in Appendix F. All simulations related to outward dike expansion were conducted using the highest available discharge scenario, S17000, as this study primarily focuses on the water level response under stress-test conditions. Multiple variants have been compiled, to investigate the effect of outward dike expansion and to be used for calibration. The spatial extent of each measure, in terms of river kilometre range, is outlined in Appendix F. For reference, Table G.1 from that appendix is repeated below.

Table G.1: Model schematisation variants for outward dike expansion analysis

Variant	Start intervention	Intervention length (est.) [km]	Outward expansion amount [m]
a1	WL_921.0	34	20
a2	WL_921.0	34	5
a3	WL_921.0	34	10
a4	WL_921.0	34	15
a5	WL_921.0	10	20
a6	WL_921.0	17	20
a7	WL_921.0	22	20
a8	WL_912.2	10	20
a9	WL_921.0	5	20

Since the outward expansions are relatively small compared to the total conveyance area, the resulting changes in absolute water levels are minimal. Therefore, the analysis focuses on water level differences relative to the baseline simulation (S17000), as these provide the necessary resolution to assess the hydraulic impact of the measures The analysis of the 1D model is also based on water level difference. The maximum water level difference due to outward expansion of variants a1 till a4 are shown in Figure G.3. The maximum computed water level difference in combination with the associated outward expansion magnitude will be used to calibrate the 1D model.

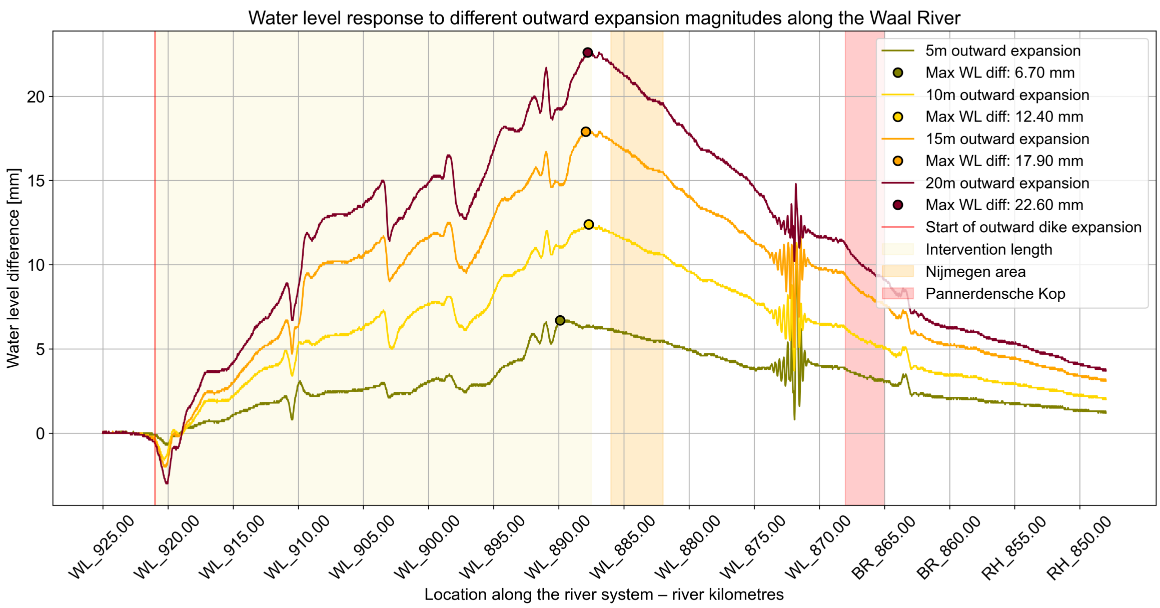

Figure G.3: D-Hydro simulation of Outward dike expansions variants a1 to a4

As shown in Figure G.3, a numerical instability appears around river kilometre WL_872. This instability is likely caused by a local increase in water level at that location, combined with a grid node in the schematisation that fails to remain numerically stable. This issue is further illustrated in Figure G.4. This error seems to introduce a pulse in the water level, as shown in Figure G.4.

It is important to note that the baseline schematisation was not modified in this region; the instability is therefore presumed to originate from the original model configuration provided by Rijkswaterstaat. All other variants exhibit similar instabilities near this river kilometre, although the effect on the water level difference appears to be more pronounced in cases with smaller absolute differences.

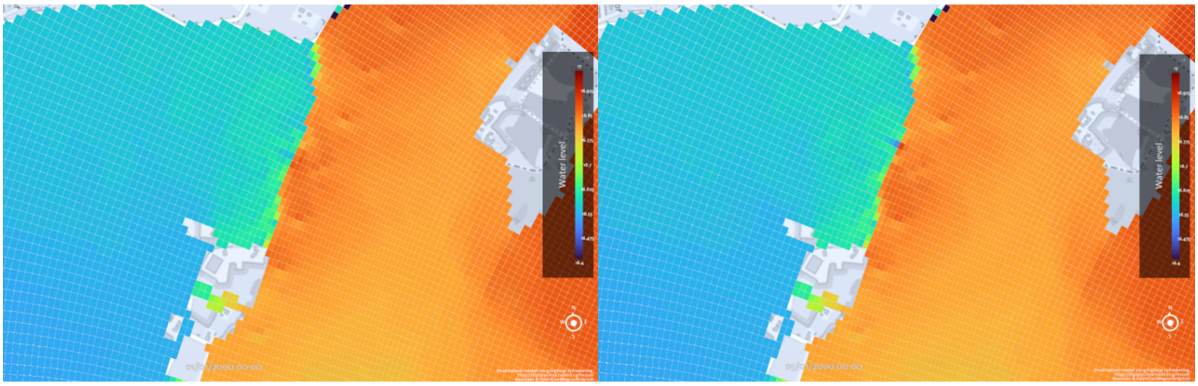

Figure G.4: D-Hydro variant simulations instability. Figure computed with Digitalys by Haskoning ©

The calibration of the 1D hydraulic model is based on two key performance criteria that the model must accurately reproduce:

1. The relationship between water level and discharge
2. The water level response resulting from outward dike expansion (ODR).

In both cases, the outputs of the 1D model are compared with those from the D-Hydro simulations, as previously discussed, using the Root Mean Squared Error (RMSE) to quantify deviations. The parameter configuration that results in the lowest combined RMSE is considered the best-fitting model setup, provided that the bias remains close to zero. Both criteria are weighted equally, as their respective performances are deemed equally important. The dimensions of both criteria remain fixed and are not scaled to eachother, while after inspections this did not give a better fit.

To reflect the importance of flood safety, higher discharges are assigned greater weight in the RMSE calculation. This weighting follows a Gaussian distribution, with the highest possible Waal discharge receiving a weight of 3, decreasing to a weight of 1 with a standard deviation of 3000 m^3/s. A similar approach is applied to the ODR analysis: the largest dike expansion is assigned a weight of 3, tapering to 1 with a standard deviation of 15 metres. This ensures that the model performs particularly well under conditions most relevant to flood risk, while still maintaining reasonable accuracy under lower-flow conditions.

Additionally, under high-flow conditions an extra penalty is applied to underestimations of water levels. This penalty is also modelled using a Gaussian distribution, but is applied exclusively to negative residuals. In all other cases, the original weighting scheme remains unchanged. The peak of this penalty distribution coincides with that of the weighting distribution with a maximum weight of 5, but with a smaller standard deviation: 1500 $^3/s$. The penalty function was not based on existing references, but was shaped through iterative testing and visual analysis during the modelling process, to encourage the model to adopt to a more conservative stance under critical conditions. No explicit penalty was applied to the outward expansion fit for underestimation, since this effect is inherently captured through the water level underestimation.

The model parameters varied during calibration include: the Chezy coefficient for both the main channel (Cc) and the floodplain (Cf), channel depth (dc), channel width (Bc), and floodplain width (Bf). The bounds within which these parameters are varied represent realistic characteristics of the Waal River, although slightly extended for possible better fitting. Step sizes were deliberately chosen to avoid excessive fitting detail, acknowledging that these parameters naturally vary at local scales and that such fine-tuning would imply a level of certainty the model cannot justifiably claim. The variables and ranges are presented in Table G.2.

Table G.2: Variables used in one-dimensional hydraulic model

Variable	Symbol	Unit	Range	Calibration step size	Fixed value	Source / Basis of estimate
Discharge	Q	m^3/s	0–12,000	–	12,000	De Vriend et al. (2017)
River bed slope	i_b	m/m	–	–	1.05×10^{-4}	Domhof et al. (2018)
Channel depth	d_c	m	5–10	0.10	8.2	Appendix H
Channel width	B_c	m	240–300	5	265	Warmink et al. (2013), Google Earth, Appendix H
Channel Chézy	C_c	$\sqrt{m}/s$	35–55	1	45	Warmink et al. (2013), Booij, van der Kaaij, Appendix H
Floodplain width	B_f	m	500–3500	100	2100	Warmink et al. (2013), Google Earth, Appendix H
Floodplain Chézy	C_f	$\sqrt{m}/s$	4–41	1	22	Warmink et al. (2013), Rijkswaterstaat Waterdienst, Grontmij Nederland BV (2009), Appendix H
Intervention length	–	m	0–70,000	–	20,000	Realistic project length

While it is recommended that "in the application of models of compound river channels, the ratio between the main channel roughness and the floodplain roughness is subject to calibration instead of calibration on both the main channel and flood plain roughness separately" [Warmink et al., 2013, p. 314], this is applied. After applying the calibration, the best 10 fits have been determined and are shown in Table G.3.

Table G.3: Top 10 best-fitting parameter combinations based on RMSE and bias

RMSE	**Bias**	**Bc**	**dc**	**Cc**	**Bf**	**Cf**
0.407	-0.022	265.0	8.20	45.0	2100.0	22.0
0.408	-0.023	260.0	8.20	46.0	2100.0	22.0
0.408	-0.020	290.0	8.20	41.0	2100.0	22.0
0.408	-0.024	285.0	8.20	42.0	2100.0	22.0
0.409	-0.019	270.0	8.20	44.0	2100.0	22.0
0.409	-0.024	255.0	8.20	47.0	2100.0	22.0
0.411	-0.025	300.0	8.20	40.0	2100.0	22.0
0.411	-0.025	250.0	8.20	48.0	2100.0	22.0
0.411	-0.025	240.0	8.20	50.0	2100.0	22.0
0.411	-0.025	245.0	8.20	49.0	2100.0	22.0

From Table G.3, it can be observed that the parameters Bc and Cc are particularly sensitive, which is expected given their direct influence on channel conveyance. In contrast, variations in dc, Bf, and Cf result in more consistent fits, suggesting these parameters are more robustly calibrated. This is especially relevant, as they represent floodplain characteristics that are critical when assessing the impact of dike expansion.

The calibration process included weights and penalties to emphasise high-flow conditions. To evaluate potential overfitting, a comparison was made with an unweighted calibration. As shown in Table G.4, the resulting parameter sets are largely consistent, with only minor differences. The most notable change is in the value of Cf, which likely explains the slight underestimation of peak discharges in the unweighted case. This suggests that the applied weighting did not introduce significant bias or overfitting and can therefore be considered a justified and acceptable modelling choice.

The parameter set from the first row of Table G.3 was selected for the final model calibration. The comparison between the 1D model and D-Hydro simulations using this configuration is shown in Figure G.5.

Due to the computational intensity of D-Hydro simulations and the additional time required for adjustments to the schematisation, only a limited number of runs were feasible for calibration. Moreover, the number

Table G.4: Top 5 best-fitting parameter combinations without weights and penalties

RMSE	Bias	Bc	dc	Cc	Bf	Cf
0.412	-0.021	275.0	8.40	44.0	2000.0	25.0
0.412	-0.020	295.0	8.40	41.0	2000.0	25.0
0.412	-0.022	265.0	8.20	45.0	2100.0	22.0
0.413	-0.023	270.0	8.40	45.0	2000.0	25.0
0.413	-0.023	260.0	8.20	46.0	2100.0	22.0

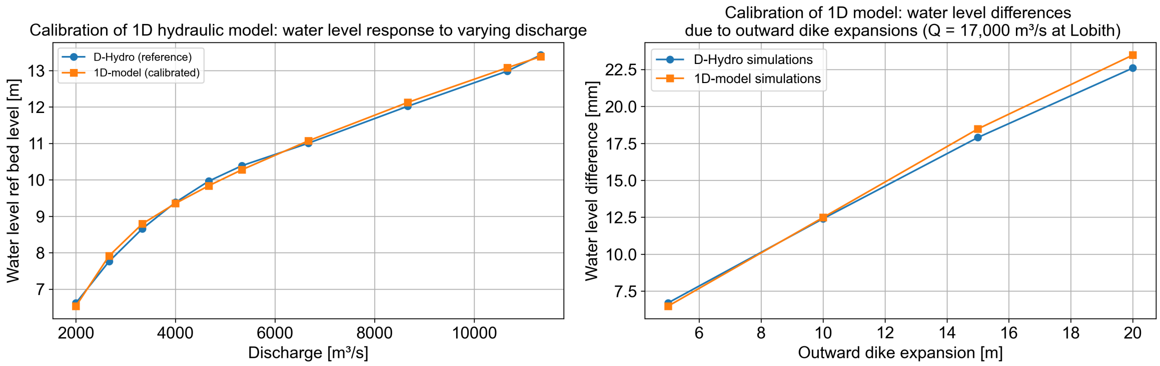

Figure G.5: Fitting of 1D model with best fitted parameters

of discharge scenarios available was insufficient to fully constrain the overall dimensions of the 1D model. Flow regime characteristics can vary significantly between rivers, leading to different water level responses; therefore, cross-validation using another river system would not be meaningful. As a result, formal validation of the model was not possible. However, as illustrated in Figure G.5, the close agreement between the simplified 1D model and the reference D-Hydro model across the two tested performance criteria supports the reliability of the model for scenario analysis within the specific context of the Waal River, as required for this study.

Backwater curve calibration

In addition to calibrating the simplified 1D compound channel model, it is also important to ensure a reasonable representation of backwater effects resulting from outward dike expansion. In reality, channel dimensions are not static, and their variability can significantly influence water level responses. The impact of different intervention lengths on water levels, schematised in D-Hydro using variants a1, a5, a6, a7, and a9, is illustrated in Figure G.6.
Although intervention lengths for the various D-Hydro variants have been estimated in the Baseline schematisation and are shown in the legend of Figure G.6, the dike trajectory does not correspond one-to-one with the river axis. Therefore, the intervention length is defined as the distance between the start of the intervention, identified at river kilometre WL_921, and the location of the maximum water level difference. It is assumed that the intervention ends where there is no more water level increase.
This definition excludes variant a8, where the intervention begins further upstream and ends in a floodplain area. In this case, the end of the intervention cannot be clearly identified, and the water level response is less distinct. Therefore, this variant is omitted for the calibration process.

To estimate backwater effects in the 1D model, an empirical fit to the Bresse backwater curve is used. To align the 1D model with the D-Hydro results, a scaling factor is applied to the backwater length, as described in Section 3.1.1. This factor influences the spatial extent of the backwater effect and, consequently, the slope of the water level response. The Bresse-based function is evaluated using the best-fitting parameters from the calibrated 1D compound channel model, which has been shown to represent the Waal River regime effectively.

As previously noted and illustrated in Figure G.4, a clear instability occurs around WL_872. This instability

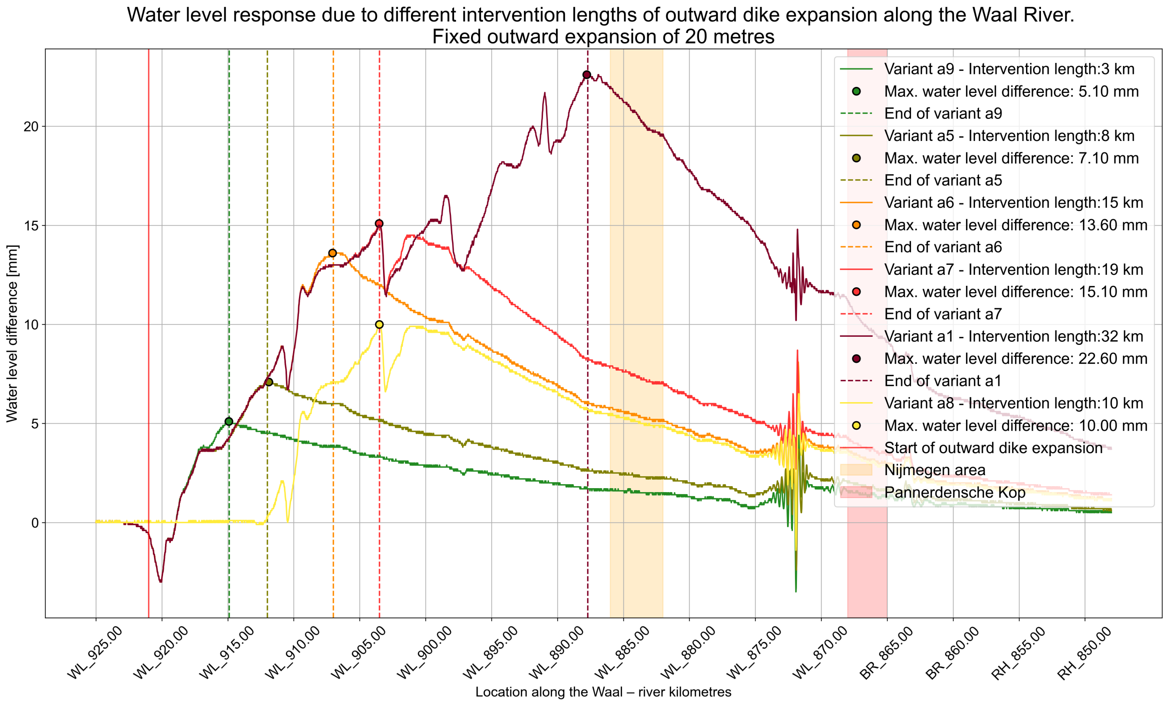

Figure G.6: Water level response due to different intervention lenghts of otuward dike expansion

significantly affects the water level response and artificially increases water levels in the shorter intervention variants, even in the absence of any physical intervention. Consequently, the adaptation length cannot be reliably derived directly from the raw D-Hydro data. Furthermore, the D-Hydro domain does not extend far enough upstream to confirm that backwater effects have fully dissipated.
To still obtain a meaningful estimate of the adaptation length from the D-Hydro simulations, an exponential decay function is fitted to the water level difference data between the end of the intervention and the onset of the instability. The fitting function is shown in Equation G.1.

$$f(x) = A \cdot \exp\left(-\left(\frac{x}{\lambda}\right)^{\alpha}\right) + C_{\text{fixed}} \tag{G.1}$$

Here, the parameter α controls the steepness of the decay, allowing the slope of the backwater effect to be adjusted during fitting. $C_{\{}$fixed resembles the asymptote and is set to zero, while the backwater effects should dissipate. The fit is performed using ordinary least squares regression of a Python function, and the best-fitting parameters are determined for each variant individually.

Within the stable region, the D-Hydro data is interpolated using this exponential function and then extrapolated beyond the instability. This extrapolation allows the backwater curve to be extended into a hypothetical upstream domain of the Rhine. The adaptation length is then defined as the distance from the end of the intervention to the point where the extrapolated curve falls below a predefined threshold, as discussed in Appendix D. It is important to note that this extrapolated domain assumes the same hydraulic characteristics as the Waal, which may introduce bias in the estimated adaptation length. The extrapolated fits and the corresponding adaptation lengths for each variant are presented in Figure G.7.
The quality of the fit is assessed visually for each variant. For shorter interventions, the fit is primarily evaluated in the region before the instability, while for longer interventions, the tail of the curve is more influential in determining the adaptation length. This visual inspection is based on the modeller's interpretation of which parts of the curve are most relevant and reliable. While this introduces a degree of subjectivity, it reflects a practical approach given the limitations of the available data and the presence of instability in the D-Hydro results.

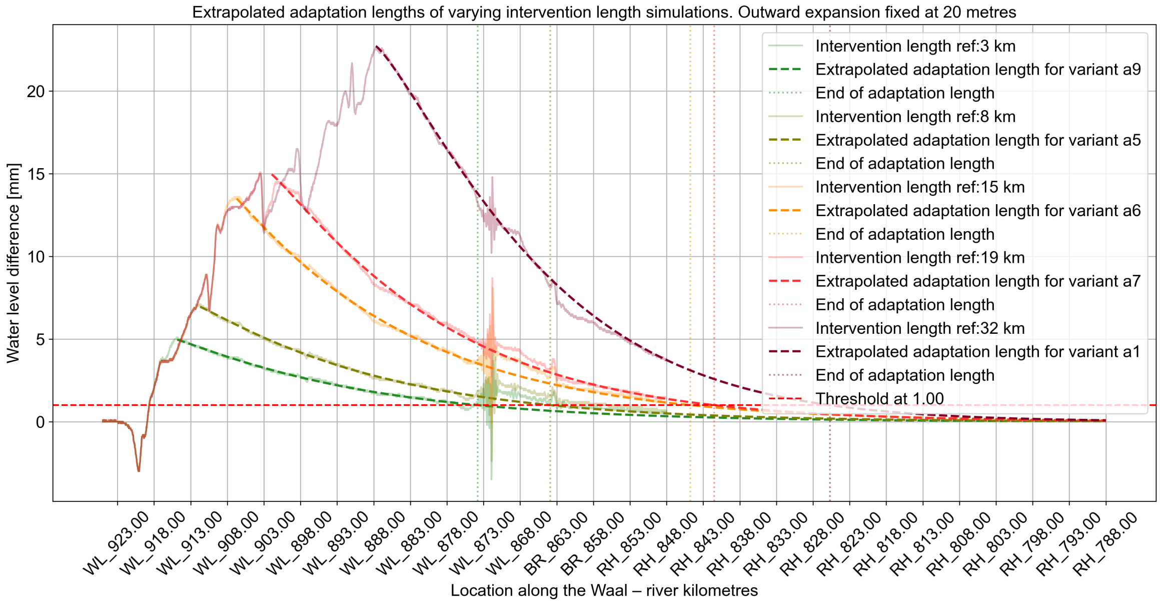

Figure G.7: Adaptation length estimation per variant using exponential fitting of D-Hydro water level differences, excluding unstable regions

The fitting procedure is sensitive to the defined end of the intervention. For example, in variant a7, where the intervention ends near the start of a floodplain, a dip in the water level difference caused the extrapolation to behave unrealistically. To address this, the end of the intervention was manually set to WL_901.95 to improve alignment with the expected curve. However, this manual adjustment may introduce overfitting and should be interpreted with caution.

Using the computed water level differences for each intervention length and the estimated adaptation lengths per intervention length, the same root mean squared error (RMSE) approach as applied to the compound channel calibration is used to identify the optimal calibration factors for the empirical Bresse-based fit. This evaluation is performed separately for the backwater effects during the intervention zone and those occurring in the adaptation zone. The RMSE and bias of the calibration factors are shown in Table G.5. The backwater effects with the best fitted calibration factors compared to the D-Hydro simulations are plotted in Figure G.8.

Table G.5: Model performance metrics for intervention and adaptation lengths

	RMSE	Bias	Calibration factor
Intervention length	1.952	0.100	1.49
Adaptation length	3404.4	0.004	1.90

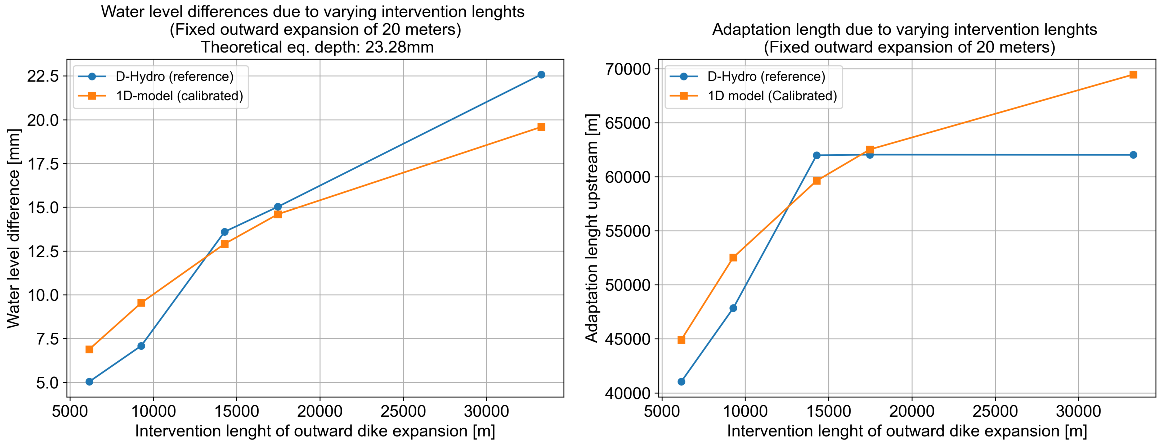

Figure G.8: Fitting of backwater effects estimations with the best fitted calibration factors

It can be concluded that both the RMSE and bias of the fits are significantly larger than those obtained from the 1D compound channel calibration. This is largely attributable to the fact that the equilibrium depth used in the Bresse fit is itself an approximated value, while fitted with a low RMSE and bias, and does not fully reflect reality. In particular, local disturbances such as bottlenecks and floodplains exert a greater influence on the estimation of the backwater curve, as evidenced by the intervention length issue observed in the A8 variant. Although the calibration factors introduce a slight adjustment that better symbolises the backwater effects in the Waal, it is important to recognise that this remains a generalised observation. Despite the relatively poor fit, this adjustment still provides a meaningful indication of the backwater response, especially given the limited data range. Since the available data do not extend beyond the point where backwater effects fully dissipate, it is not possible to draw firm conclusions or perform a robust calibration of the adaptation lengths. Consequently, the conclusions drawn from these approximations are only valid for this specific section of the Waal. These findings highlight the high sensitivity of the calibration process to local floodplain water storage, which significantly influences both the intervention and adaptation lengths. This underscores the importance of accounting for spatial heterogeneity when interpreting model performance and drawing conclusions.