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Lendering, Kasper

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ADVANCING METHODS FOR EVALUATING FLOOD RISK REDUCTION MEASURES

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by:

Kasper Theo LENDERING
Master of Science in Civil Engineering, Delft University of Technology, the Netherlands
born in Willemstad, Curaçao

This dissertation has been approved by the promotors:

Composition of the doctoral committee:

Rector Magnificus	chairman
Prof.dr.ir. S.N. Jonkman	Delft University of Technology, Promotor
Prof.dr.ir. M. Kok	Delft University of Technology, Promotor

Independent members:

Prof.dr.ir. P.H.J.A.M. van Gelder	Delft University of Technology
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Ir. R. Jorissen	Rijkswaterstaat, the Netherlands

Other members:

Em.prof.drs.ir. J. K. Vrijling	Delft University of Technology

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Summary

Throughout history, societies have shown increasing vulnerability to flood events (e.g. flash floods, river floods and storm surge). With studies suggesting that the frequency and intensity of flood events increasing, coupled with growing urbanization in flood prone-areas, both human exposure and economic damages due to flooding continue to rise worldwide. To mitigate evolving flood risk, existing flood defence systems (e.g., levees, dikes, reservoirs and dams) will need to be adapted and/or new systems designed and built. Besides more traditional flood defence measures, interventions that aim to reduce the potential consequences of flooding are also gaining momentum; examples include spatial planning, the use of emergency measures, temporary flood barriers and green roofs.

The application of risk-based approaches for the design of flood risk reduction systems has become increasingly common in flood management. While this approach is often used to assess the risk and reliability of more traditional flood defences, they have not been applied or operationalised for the previously described other types of interventions. As a result, decision makers are not able to assess the effectiveness or performance of these innovations when included in a risk reduction system.

To fil this void, this dissertation aimed to advance the risk-based approach for flood risk reduction interventions to allow for assessing the risk and reliability associated with specific interventions. Starting with advancing existing methods for reliability analysis of flood defences to assess the failure probability of canal levees and the effectiveness of emergency measures for flood prevention. In addition, existing cost benefit analysis methods for flood defences are advanced to enable optimization of different flood risk reduction strategies, such as land fills and flood defences, depending on the size of the area protected and its land use. Finally, a broader risk-based analysis is proposed analyses the effectiveness of flood adaptation innovations applied in different layers of flood risk reduction systems.

Canal levees are mainly earthen levees along drainage canals that drain excess water from polders to the main water bodies. To quantify the failure probability of canal levees, and gain insight in the risks associated with these systems, several extensions to existing statistical models have been developed. These extensions include a method to account for water level regulation in canals, the effect of maintenance dredging on the geohydrological response of the canal levee and performing a posterior analysis to account for survived loads in the past (Bayesian Updating). The posterior analysis opens opportunities for testing the resistance of a canal levee under different combinations of loads. The results of a case study demonstrate that the proposed approach can be used to quantify the probability of failure of canal levees. With these methods, it is possible to evaluate and prioritize different flood risk reduction measures (e.g., levee reinforcement or increasing drainage capacity) in terms of their costs and benefits (or risk reduction).

Emergency measures, defined as temporary measures implemented during a (threatening) flood to reinforce, or repair, damages in flood defences and prevent breaching, were also considered in this dissertation. Examples include placing sand bags on top of flood defences to gain more height or constructing a soil berm against flood defences for more horizontal stability. To evaluate the effectiveness of emergency measures for flood prevention, this dissertation developed a method that includes organizational, logistical and technical failure of emergency measures in the overall reliability analysis of flood defences. Based on a case study of overflow and piping emergency measures, it became clear that the probability of the flood defence system can be reduced by applying emergency measures. The probability of human errors and logistical failure proved to be dominant, compared to technical failure probability of the measure. An analysis of the costs and risk reduction obtained with emergency measures showed that these measures are far less cost-effective on the long term than permanent reinforcements. However, emergency measures could play a role as an interim solution before permanent reinforcements are finished.

This dissertation introduced a method to optimize the selection of risk reduction strategies. The method expands existing economic optimization approaches for flood defences, by introducing (largely) analytical formulations to include the effects of land fills or other approaches to limit flood consequences. The method considers the size of the protected area and associated damages, the costs and sizing of interventions and the corresponding likelihood of flooding. Several practical examples were discussed. Overall, the cases demonstrate a strong preference for flood defences over fills, given high costs and large protected areas. Fills are preferred for small areas and/or for low marginal cost. A combination is preferred when the value protected by the flood defence is low and the value protected by the fill is high or when the high value development is relatively small in size. The sensitivity of outcomes to the choice of the main input parameters is presented, and implications for the selection of strategies in developing and developed countries are discussed. Additional factors that affect the selection of strategies are discussed, such as the need to include water drainage for areas protected by flood defences, time and budget constraints and governmental context. Overall, the methods developed in this dissertation aim to support decision makers in developing optimal strategies to manage and reduce flood risk.

Using the insights and models developed in this dissertation, a framework is proposed to assess the performance of flood adaptation innovations within the risk-based approach. Flood adaptation innovations are defined as solutions that have not been assessed in terms of risk reduction and/or reliability, or solutions that have not yet been applied in practice. Examples include temporary flood barriers, green infrastructure and early flood warning systems. Four performance indicators are proposed that allow for evaluating the performance of these solutions within the risk-based approach: effectiveness, durability, reliability and costs. By assessing the performance of each indicator, end-users can compare different types of innovations and make risk-informed decisions about their implementation. The practical application of the framework is demonstrated for three examples of innovations in a case study considering pluvial flooding. The following measures were considered: an early flood warning system, a green roof, and a temporary flood barrier (no permanent flood barriers were considered in the comparison). In the example, temporary flood barriers proved to be most effective, followed by green roofs and an early flood warning system.

The principles and methods developed and applied in this dissertation may be used for challenges in flood risk management which have not been described specifically in the respective chapters. Similarly, methods for assessing the effectiveness of emergency measures or flood adaptation innovations can also be used for measures or innovations for other hazards (e.g., wildfires or extreme weather). Finally, the methods for optimizing flood risk reduction strategies can also be applied to different floodproofing measures (e.g., raising houses) and is relevant for different areas subject to flood risks around the world (e.g., the Vietnam deltas or Japan coasts).

While the risk-based approach is often deemed complex and expensive; this dissertation demonstrates that with the development of several extensions or tools, it is possible to assess the reliability and risk of innovative interventions within the overall system of flood risk reduction. This opens opportunities to compare and evaluate innovative solutions based on reliability, risk reduction and (cost) effectiveness and aids decision makers to consider a wider range of interventions for flood risk reduction. The case studies and practical examples included in this dissertation have underlined this possibility and provide hands on examples of the frameworks and methods developed. Using these methods, decision makers will gain a better understanding of the risk-reduction system and how it performs, ultimately providing the necessary input and information for substantiating decisions regarding flood risk reduction.

Samenvatting

De kwetsbaarheid voor overstromingen (bijvoorbeeld door extreme regenval, hoogwater op rivieren of stormvloed) van samenlevingen over de hele wereld is groot. Recent onderzoek suggereert dat de frequentie en intensiteit van overstromingen toenemen terwijl ook de verstedelijking in overstromingsgevoelige gebieden steeds verder toeneemt. Met als gevolg dat de blootstelling van mensen aan overstromingen en de potentiele economische schade door overstromingen wereldwijd zal blijven toenemen.

Om toenemende overstromingsrisico's te mitigeren worden bestaande waterkeringen (bijv. dijken, dammen en stormvloedkeringen) aangepast en/ of nieuwe systemen ontworpen en gebouwd. Naast de meer traditionele, permanente, ingrepen als dijken en stormvloedkeringen is er tegenwoordig meer aandacht voor maatregelen gericht op het het verkleinen van de potentiele gevolgen van overstromingen; voorbeelden zijn ruimtelijke ordening, de inzet van noodmaatregelen, tijdelijke waterkeringen en groene daken.

Het toepassen van de risico-gestuurde aanpak voor het ontwerp van waterkeringssystemen is door de jaren heen steeds gebruikelijker geworden. Hoewel deze benadering tot op heden vooral is ingezet om de betrouwbaarheid en risico's van meer traditionele ingrepen in het systeem te beoordelen, is deze niet gebruikt of geoperationaliseerd voor andere type maatregelen. Met als gevolg dat besluitvormers niet in staat zijn de effectiviteit van deze maatregelen te vergelijken aan de meer traditionele maatregelen. Met effectiviteit wordt bedoeld de mate waarin de maatregel in staat is het risico op overstromen te reduceren.

Dit proefschrift is gericht op het bevorderen van de risico-gestuurde aanpak voor waterkeringen zodat de betrouwbaarheid en risico's van specifieke interventies in een systeem beoordeeld kunnen worden. Daartoe ontwikkelt dit proefschrift een methode om de betrouwbaarheid van boezemkaden (langs boezemkanalen) en de effectiviteit van noodmaatregelen voor het voorkomen van overstromingen (bijv. zandzakken) te kwantificeren. Daaropvolgend is een methode ontwikkeld om de hoogte van terpen en waterkeringen te optimaliseren, afhankelijk van de oppervlakte van het te beschermen gebied, het landgebruik en de waarde van dat gebied. Tot slot is een aanpak voor een bredere risicoanalyse voorgesteld waarmee de prestaties van "overstromingsadaptatie-innovaties" gemeten kan worden. Overstromingsadaptatie-innovaties zijn gedefinieerd als oplossingen waarvan de effectiviteit niet is beoordeeld in termen van betrouwbaarheid en risico's en/ of oplossingen die nog niet toegepast zijn in de praktijk.

Boezemkaden bestaan uit grondlichamen langs afwateringskanalen die overtollig water uit polders afvoeren naar het buitenwater: een meer, rivier of zee. Om de faalkans van boezemkaden te kwantificeren, en inzicht te krijgen in het overstromingsrisico in deze systemen, zijn verschillende uitbreidingen van bestaande

probabilistische modellen om faalkansen van dijken te bepalen ontwikkeld. Deze uitbreidingen omvatten een methode om rekening te houden met i) de waterstandregulatie in boezemkanalen, ii) het bepalen van de kansverdeling van het freatisch vlak, iii) het effect van onderhoudsbaggerwerk op de kans op piping en iv) het uitvoeren van een bewezen sterkte analyse om rekening te houden met overleefde belastingen uit het verleden. De resultaten van een casestudie tonen aan dat de voorgestelde aanpak in staat is de faalkansen van boezemkaden te kwantificeren. De bewezen sterkte analyse opent mogelijkheden om de sterkte van een boezemkade te testen onder verschillende combinaties van belastingen. Hiermee is het mogelijk om maatregelen gericht op het reduceren van overstromingsrisico's (bijv. dijkversterking of verhogen van afwateringscapaciteit) te vergelijken en te prioriteren op basis van kosteneffectiviteit, door de kosten te vergelijken aan de mate waarin het overstromingsrisico afneemt.

Noodmaatregelen zijn in dit proefschrift gedefinieerd als tijdelijke maatregelen die worden geïmplementeerd tijdens een (dreigende) overstroming om schade aan waterkeringen te herstellen en een bres te voorkomen. Voorbeelden zijn het plaatsen van zandzakken bovenop waterkeringen voor een grotere kerende hoogte of het aanleggen van een stabiliteitsberm om afschuiven te voorkomen. Om de effectiviteit van deze noodmaatregelen te bepalen, ontwikkelde dit proefschrift een methode waarin menselijk-, logistiek- en technisch falen van noodmaatregelen meegenomen worden in de betrouwbaarheidsanalyse van waterkeringen. Zo ontstaat inzicht in de bijdrage van noodmaatregelen aan het verlagen van de faalkans van de dijk. Op basis van een casestudie van noodmaatregelen voor de faalmechanismen overslag en piping is geconstateerd dat de faalkans van een dijkring beperkt gereduceerd kan worden met noodmaatregelen. De kans op menselijke fouten en logistiek falen bleek dominant in vergelijking tot de technische faalkans van een noodmaatregel. Een analyse van de kosten en baten (uitgedrukt in een verlaging van het overstromingsrisico) van noodmaatregelen toonde aan dat deze maatregelen veel minder kosteneffectief zijn dan permanente versterkingen, maar wel een rol zouden kunnen spelen als tussentijdse oplossing.

Dit proefschrift beschrijft een methode om de sommatie van de investeringskosten en de netto contante waarde van het overstromingsrisico, over de gehele levensduur van de maatregel, te minimaliseren. De oppervlakte van het te beschermen gebied, het landgebruik en de waarde daarvan zijn als variabelen meegenomen in deze methode, om inzicht te geven in de invloed van deze variabelen op het optimale beschermingsniveau. Verschillende praktische voorbeelden zijn beschouwd, waaruit in het algemeen blijkt dat waterkeringen economischer zijn dan terpen bij grote oppervlakten van het te beschermen gebied. Terpen zijn alleen economischer voor kleine oppervlakten of wanneer de marginale kosten van terpen veel lager zijn dan die van waterkeringen.

Dit proefschrift beschrijft een methode voor het optimaliseren van portefeuilles (of combinaties) van maatregelen voor het reduceren van overstromingsrisico's. De methode bouwt voort op bestaande optimalisatiebenaderingen voor waterkeringen, door (grotendeels) analytische oplossingen af te leiden om de effecten van terpen (of andere schade mitigerende maatregelen) mee te nemen. De methode houdt rekening met de kosten van de beschouwde maatregelen, de omvang van het te beschermen gebied en de potentiele schade bij een overstroming. Verschillende praktische voorbeelden zijn beschouwd, waar in het algemeen uit blijkt dat waterkeringen economischer zijn dan terpen. Terpen hebben enkel de voorkeur voor kleine gebieden of wanneer de aanleg van terpen laag zijn. Een combinatie van waterkeringen en terpen heeft de voorkeur wanneer de waarde van het gebied beschermd door de waterkering laag is in vergelijking tot de waarde beschermd door de terp. Desondanks blijken terpen (of vergelijkbare maatregelen als het bouwen op palen) regelmatig ingezet te worden. Andere redenen kunnen dit verklaren, bijvoorbeeld het een beperkt budget of bouwtijd. Door te bespreken hoe deze redenen besluitvorming beïnvloeden, helpt dit proefschrift bij het vergelijken en evalueren van verschillende strategieën.

Aan de hand van de inzichten en modellen die in dit proefschrift zijn ontwikkeld, is een kader voorgesteld om “overstromingsadaptatie-innovaties” binnen de risico-gestuurde benadering te beoordelen Voorbeelden van dergelijke innovaties zijn tijdelijke waterkeringen, groene daken en vroegtijdige waarschuwingssystemen voor overstromingen en wateroverlast. Vier prestatie-indicatoren zijn voorgesteld die het mogelijk maken om dergelijke oplossingen te beoordelen binnen de risico-gestuurde aanpak: effectiviteit, duurzaamheid, betrouwbaarheid en kosten. Door de prestaties van iedere indicator te beoordelen, kunnen eindgebruikers verschillende soorten innovaties vergelijken en gefundeerde beslissingen nemen met betrekking tot hun implementatie. De praktische toepassing van het raamwerk wordt gedemonstreerd voor drie voorbeelden van innovaties tegen overstromingen als gevolg van hevige regenval: een vroegtijdig waarschuwingssysteem voor overstromingen, een groen dak en een tijdelijke waterkering. Van de beschouwde voorbeelden hebben tijdelijke waterkeringen de grootste kosten-batenverhouding, gevolgd door de groene daken en het vroegtijdig waarschuwingssysteem.

De principes en methoden die in dit proefschrift zijn ontwikkeld en toegepast, kunnen tevens gebruikt worden andere dan in dit proefschrift beschreven uitdagingen binnen waterveiligheid. Zo kunnen de methoden voor het beoordelen van de effectiviteit van noodmaatregelen of overstromingsadaptatie-innovaties ook worden gebruikt voor maatregelen bedoeld voor andere gevaren (bijv. bosbranden of extreme neerslag). Ook de methoden ontwikkeld voor het optimaliseren van strategieën voor overstromingsbescherming kunnen worden toegepast op andere typen maatregelen (bijvoorbeeld het verhogen van huizen) en is relevant voor verschillende gebieden die onderhevig zijn aan overstromingsrisico's over de hele wereld (bijvoorbeeld de Vietnam-delta's of de Japanse kusten).

Hoewel de op risico-gestuurde aanpak voor waterkeringen vaak als complex en duur wordt ervaren, laat dit proefschrift zien dat het met de ontwikkeling van verschillende uitbreidingen of hulpmiddelen mogelijk is om de betrouwbaarheid en risico’s van verschillende (innovatieve) interventies binnen het totale systeem te beoordelen. Deze methoden maken het mogelijk interventies in een systeem te vergelijken en evalueren op basis van betrouwbaarheid, risicoreductie en (kosten) effectiviteit, zodat besluitvormers een breed scala aan (innovatieve) oplossingen kunnen beschouwen voor het reduceren van overstromingsrisico’s. De praktijkvoorbeelden en casestudies in dit proefschrift hebben dit nog eens benadrukt en helpen besluitvormers met de vergelijking en implementatie van dergelijke interventies. Met behulp van deze methoden krijgen zij een beter begrip van het systeem en hoe het presteert, inclusief eventuele interventies. Daarmee draagt dit proefschrift bij aan de benodigde input en informatie voor het onderbouwen van beslissingen voor het reduceren van overstromingsrisico’s.

Contents

1

Introduction

Throughout history, societies have shown increasing vulnerability to flood events (e.g. flash floods, river floods and storm surge) (Bouwer 2011). Globally floods cause enormous economic damage and loss of life every year. Between 1980 and 2014, flood-related damages accounted for 36% of all losses from natural disasters (Hoeppe 2016) while in the last century, floods killed about 100,000 humans (Jonkman 2005). Recent climate observations suggest that the frequency and intensity of flood events are increasing resulting in larger flood hazards and less lead time prior to an event (e.g., due to increasing precipitation intensities, higher storm surges and sea level rise) (EEA 2012a; R. S. Kovats et al. 2014). Coupled with growing urbanization in flood-prone areas – especially along coastlines and in river deltas – human exposure to floods (i.e., potential for loss of life) and flood damages are also rising. These trends are expected to continue to increase during the 21st century (IPCC 2014) and lead to an increase of flood risk (Hallegatte et al. 2013).

To mitigate evolving flood risk, existing flood defence systems (e.g., levees, dikes, reservoirs and dams) will need to be adapted and/or new systems designed and built. Additionally, flood risk can be mitigated by increasing adaptive capacity of flood prone areas, for example by increasing drainage capacity, 'flood proofing' buildings (i.e., adapting structures to reduce or eliminate potential flood damages), implementing temporary or emergency protective measures (e.g., sand bags or temporary flood barriers) and/or through flood warning and evacuation. A rational evaluation of various interventions is required for decision making within flood risk management.

1.1 The risk-based approach for flood management: a brief description

Risk-based approaches are commonly used to assess flood risk and evaluate risk reduction measures based on cost-effectiveness (Vrijling 2001; Jonkman & Kok 2008a). While the definition of risk varies across different disciplines, herein risk is defined as the product of the annual probability (i.e., likelihood) of flooding and its potential adverse consequences, where consequences are a function of exposure to, for example, people, buildings, business, and infrastructure, and their vulnerability (i.e., engineering, economic, social, environmental vulnerability) (Cardona et al. 2012; Traver 2014; Klijn et al. 2015). Following this definition, risk reduction can be achieved by reducing either the flood probability (e.g., by increasing drainage capacity or reinforcing flood defences) or its potential consequences (e.g., by raising or floodproofing buildings).

Over recent decades, significant progress has been made in developing comprehensive risk-based frameworks for assessing the risks and reliability associated with flood defence systems (Schweckendiek 2015; Morales-Nápoles et al. 2014; Gelder 2000; Rijkswaterstaat 2016; Schweckendiek & Vrouwenvelder 2013; Vrijling 2001). Here, reliability is defined as the likelihood, or probability, that a system performs as intended. Decision makers (e.g., water boards or governments) use these frameworks to make risk-informed decisions concerning interventions in flood defence systems (mainly focused on dikes and levees) based on costs and benefits (i.e., risk reduction). Here, costs are determined by the initial investment and operation and maintenance cost of the intervention over its lifetime, whereas the benefits are expressed as flood damages avoided and/or a reduction of risk to life (Jonkman 2007).

1.1.1 The traditional safety-oriented approach

Traditionally, flood risk management is based on a safety-oriented approach in which structural measures (e.g., levees and storm surge barriers) are built to protect to the height of a design flood (Schumann 2017). These design levels were commonly derived using a pragmatic, safety-oriented, approach, based on historical flood events (Paul Sayers 2012). If flood levels exceeded the height of existing flood defences, or if flood defences would breach, local inhabitants would increase the height or width of the flood defence to provide more protection for future flood events. Because the probability of events larger than the design flood is small, the risk behind a structure is (generally) ignored (Ludy & Kondolf 2012). The safety-oriented approach is currently used as the basis for decisions regarding flood mitigation in the United States, where flood insurance is only mandatory for federally-mortgaged structures in the 100-year floodplain and areas located behind levees are removed from the floodplain maps and considered to be safe. After major flooding killed 1,836 people in the Netherlands in 1953, it became clear that more comprehensive methods for deriving safety levels and designing flood defence systems were needed. Therefore, van Dantzig developed a risk-based approach (van Dantzig 1956) to derive and assign safety standards to flood defence systems along the major rivers and coast in the Netherlands. The safety standards were derived by optimizing the cost of raising flood defences against its benefits (i.e., the damages avoided) (van Dantzig & Kriens 1960). The construction of the Dutch Delta Works, with protection levels up to the derived safety standards, resulted in a significant increase of flood safety in the Netherlands (van Dantzig & Kriens 1960).

In his approach, van Dantzig approximated the probability of flooding by estimating the likelihood that water levels would exceed the retaining height of flood defences. In other words, the probability of overflowing was used as a proxy for the probability of flooding, based on statistical descriptions of water levels (both in rivers and along the coast). This means that geotechnical failure mechanisms, which may occur before water levels overtop the flood defence, are neglected or at least assumed to have a smaller probability (Schweckendiek 2014). To justify this approximation, guidelines for the design of flood defences were developed that require significantly smaller probabilities of other (geotechnical) failure mechanisms (e.g., by constructing wide levees). This approach shows how the risk-based approach and the safety oriented come together: the risk-based approach was used to derive safety standards expressed in a required height of the flood defence.

Besides using the probability of overflowing as a proxy for the probability of flooding, van Dantzig assumed complete flood control for levels below the height of the structure, and complete loss of (economic) value for water levels exceeding the height of the structure. However, the consequences of flooding very much depend on flood depth (e.g., flood depths of 0.5 meter will result in much lower consequences than flood depths of 5 meters) (de Moel & Aerts 2011).

Another simplification used by van Danztig was to neglect time dependency. At the time of developing the considered model, the need for improving flood protection by heightening the levees was apparent, due to the recent major floods in the Netherlands (Eijgenraam 2006). However, economic growth, degradation of flood defences and sea level rise can significantly impact optimal investment strategies, which are defined by the optimal elevation level and moment in time when flood defences are reinforced. Therefore, Eijgenraam (2006) later improved the existing van Danztig model by accounting for the mentioned time dependencies.

Despite these simplifications, until very recently, the safety standards derived with the van Dantzig approach were still the basis for flood safety in the Netherlands. Furthermore, similar methods are used in the United States, where safety levels are based primarily on the quantification of a hazard for a given return period (generally 1/100 per year) based on the assumption of complete flood control. This implies that events with probabilities of 1/100 per year (corresponding to the design level of the defence) and smaller are ignored.

1.1.2 The current risk-based approach

In recent decades, the risk-based approach has been further improved with techniques and methods that enable full probabilistic analysis of the reliability of flood defences, taking the variability and uncertainty in both hydraulic loadings and strength in to account (Schweckendiek 2014). These techniques and methods allow for full probabilistic analysis of flood defences to determine their probability of flooding, considering all failure mechanisms of a flood defence. This allows for more accurate estimates of the probability of flooding, which until then was still estimated by the probability of overflow (as described in the previous section). In addition, flood simulation and damage models have been developed that allow for more accurate modelling of flood damages, depending on land use and flood depth (Kok et al. 2006).

These developments have significantly improved the ability to assess the risk and reliability associated with flood defence systems and have resulted in significant progress in the field of flood risk management. The full probabilistic risk-based approach was used to assess the risk of flooding of all flood prone areas along the rivers and coast of the Netherlands in a project called ‘Flood Risk of the Netherlands’ (Rijkswaterstaat 2016). The results of this project provided input for developing new safety standards for flood defences in the Netherlands (Rijkswaterstaat 2015), based on optimizing economic damages as well as considering risk to life (Jonkman et al. 2005; Slijkhuis et al. 2001; Jonkman 2007; Vrijling et al. 1998a; Jongejan & Maaskant 2013). The new safety standards are now expressed as a maximum failure probability of the flood defence which leads to flooding of an area, instead of the earlier explained probability of exceedance of the flood defence level.

Under the Water Act (Anon 2010), as of January 1st 2017, the new safety standards have been applied to the main flood defences in the Netherlands. With the introduction of the full probabilistic approach, more accurate insights in the actual risk and reliability levels associated with flood defence systems is possible (Schweckendiek 2015). Moreover, these methods allow for the evaluation and prioritization of interventions in flood defence systems based on their cost-effectiveness (Jongejan & Maaskant 2013).

Also outside of the Netherlands, countries have started moving towards a more risk-based approach for flood management, for example in the United States (Jonkman & Kok 2008b; NRC 2013a; NRC 2014), the UK (Hall et al., 2003) and in the Shanghai region in China (Jiabi et al. 2013). Overall, it can be observed that the insights from risk and reliability analyses are now at a stage that they can be more directly applied in policy making (e.g., safety standards) and the design and management of flood defences (Schweckendiek 2015).

1.2 Challenges of the risk-based approach

While the advantages of the methods and techniques of the full probabilistic risk-based approach are generally recognized (NRC 2013b), their understanding and application by engineers in practice is still relatively limited. Practitioners find that the full probabilistic approach is difficult to apply, partly due to the need for a profound understanding of reliability and risk, but also due to the need to perform comprehensive statistical and numerical analyses to assess probabilities of failure (Schweckendiek 2015). Additionally, the application of the approach requires good understanding and combination of the physics of structures as well as the statistical/ probabilistic characteristics of its parameters.

The full probabilistic approach was developed and has been applied to preventive structures like flood defences along rivers and coasts (and is thus straight-forward for these applications). However, it has not been widely applied (or tested) to assess risk and reliability associated with other interventions within a flood risk reduction system (see textbox). As other countries also begin to move towards utilizing risk-based approaches to mitigate the economic impacts of floods, there is a need for insight and research into the application of the risk-based approach to assess the performance of other interventions in the system. For example, more traditional risk-based approaches are currently still used to assess the reliability of canal levees. Also, the reliability and risk associated with emergency measures and other (innovative) measures for flood risk reduction have yet to be implemented within the risk-based approach.

Therefore, this dissertation addresses specific challenges of the risk-based approach in the design and optimization of flood risk reduction systems. Existing concepts for reliability analysis of flood defences will be advanced to enable quantification of the failure probability of canal levees, considering multiple loads. The following challenge includes developing models that can assess the reliability and effectiveness of emergency measures for flood prevention. In addition, based on existing optimization models for flood defences, new models will be developed that also consider other flood risk reduction strategies (e.g., land fills and floodproofing), while also including the variability in size of the system, its land use and the dependency of damages on flood depth. Finally, a broader risk-based analysis is proposed analyses the effectiveness of flood adaptation innovations applied in different layers of flood risk reduction systems. These challenges require the development of additional extensions or adjustments to the current full probabilistic risk-based approach used in flood risk management.

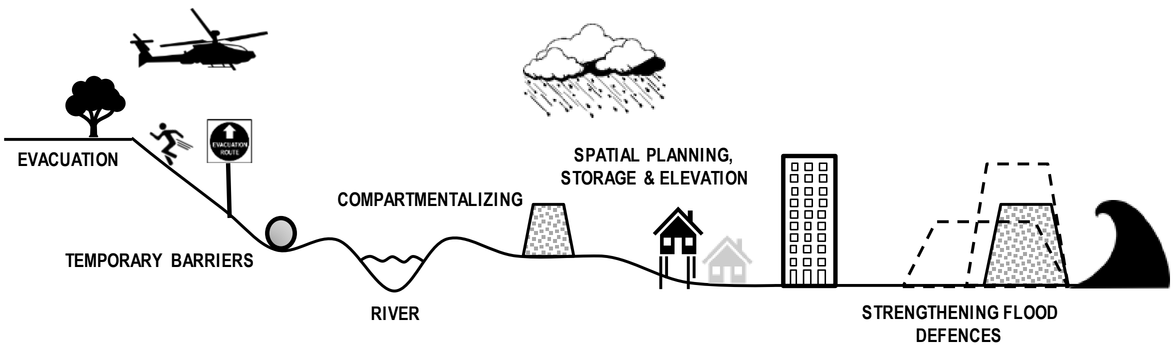

Figure 1: Flood risk management and multi-layer safety: (1) protection, (2) prevention, and (3) preparedness

Flood risk reduction systems are often conceptualized as three layers: (1) protection, (2) prevention, and (3) preparedness (Kolen & Kok 2011; Kolen et al. 2012). In this context, Layer 1 (protective) measures reduce the probability of flooding through structural measures (e.g., the flood defences and/or retention areas), whereas Layer 2 (prevention) and Layer 3 (preparedness) measures address the consequences of flooding through, for example, floodproofing of buildings, spatial planning, emergency measures and evacuation. In literature, different terms are used for these three layers. For example, layer 1 is also often named 'Prevention' while layers 2 and 3 are called 'Spatial design' and 'Crisis Management/ Emergency Preparedness' (Kok et al. 2017).

Examples of measures within each layer are included in the table and in Figure 3 (measures considered in this dissertation in **bold**):

Layer	***Examples of Measures***
Protection	*dams;* ***levees****; floodwalls;* ***dikes****; seawalls; flood gates;* ***temporary flood barriers****; floodways and spillways; channel modifications; storm water management; on-site retention; detention; breakwaters; bulkheads; groins; revetments; nourishments;*
Prevention	*spatial planning; safe land-use practices (e.g., setbacks); construction standards and building codes (e.g., vertical elevation);* ***elevating buildings****;* ***flood proofing buildings****; acquisition and relocation; coastal zone management;* ***green roofs***
Preparedness	*forecasting;* ***early warning****; evacuation;* ***emergency measures****; floodplain mapping; flood insurance; disaster relief; subsidies; public awareness and education*

Table 1: Examples of solutions for reducing flood risk by layer.

1.3 Aim of this dissertation

This dissertation aims to advance the risk-based approach by developing several additions that allow for assessing the risk and reliability associated with different flood risk reduction strategies. Specifically, this dissertation addresses the following research questions:

i) how can the probability of failure of canal levees be quantified?
ii) how can the effectiveness of emergency measures for flood prevention be assessed?
iii) how can portfolios of flood defences and land fills be optimized, considering the costs and risks associated with increasing size of the area to be protected?

iv) how can the performance of flood adaptation innovations be evaluated within the risk-based approach?

By addressing these research questions, this dissertation aims to contribute to further development and application of the risk-based approach by providing hands on examples of its use by decision makers. For this purpose, several extensions to the current refined risk-based approach are proposed and explained using practical examples in each chapter.

1.4 Originality/ contribution

The novel contribution of this dissertation is the development of additions to the risk-based approach to assess the risk and reliability associated with different flood risk reduction strategies. While existing methods and techniques for assessing risk and reliability were mostly applied to river and coastal flood defences (Jongejan et al. 2013), this dissertation expands its use to canal levees, considering pluvial flooding, and other interventions within the prevention and preparedness layers of a risk reduction system. Specifically, this dissertation advances the risk-based approach for i) quantifying the failure probability of canal levees (chapter 2), ii) assessing the effectiveness of emergency measures (chapter 3), iii) optimizing portfolios of risk reduction strategies: flood defences and/or land fills (chapter 4), iv) and assessing the performance of flood adaptation innovations (chapter 5).

For this purpose, several extensions to existing risk-based approaches are developed. Chapter 2 focusses on canal levees and develops several extensions to account for i) regulation (and drainstop) of water levels in canals, ii) the possibility of (removal of) hydraulic resistance on the bottom of the canal due to maintenance dredging, iii) the uncertainty in traffic loads and iv) the uncertainty of the phreatic surface. In addition, performance observations are used to assess the failure probability of canal levees more accurately. Chapter 3 discusses methods to include organizational and logistical failure of emergency measures within reliability assessments of flood defences. In chapter 4, a method for optimization of the elevation level of land fills and flood defences considering multiple parameters (e.g., size of the area and its land use) is developed and solved analytically. Finally, chapter 5 proposes methods to assess the performance of flood adaptation innovations within the risk-based approach.

Each chapter provides hands on examples of the proposed methods in case studies and concludes with its findings and suggestions for further research. While the additions developed in this dissertation are often based on the Dutch practice of flood risk management, these are also applicable to issues outside of the Netherlands. For this purpose, several examples are included that consider more international applications.

1.5 Dissertation overview

This dissertation further advances the risk-based approach for flood defences to allow for assessing the risk and reliability associated with specific interventions in different layers of a flood risk reduction system. The following figure displays how each chapter can be categorized within a specific layer of a risk reduction system (i.e., the protective, preventive or preparedness layer) and whether that specific chapter focusses on reliability or risk analysis (or both).

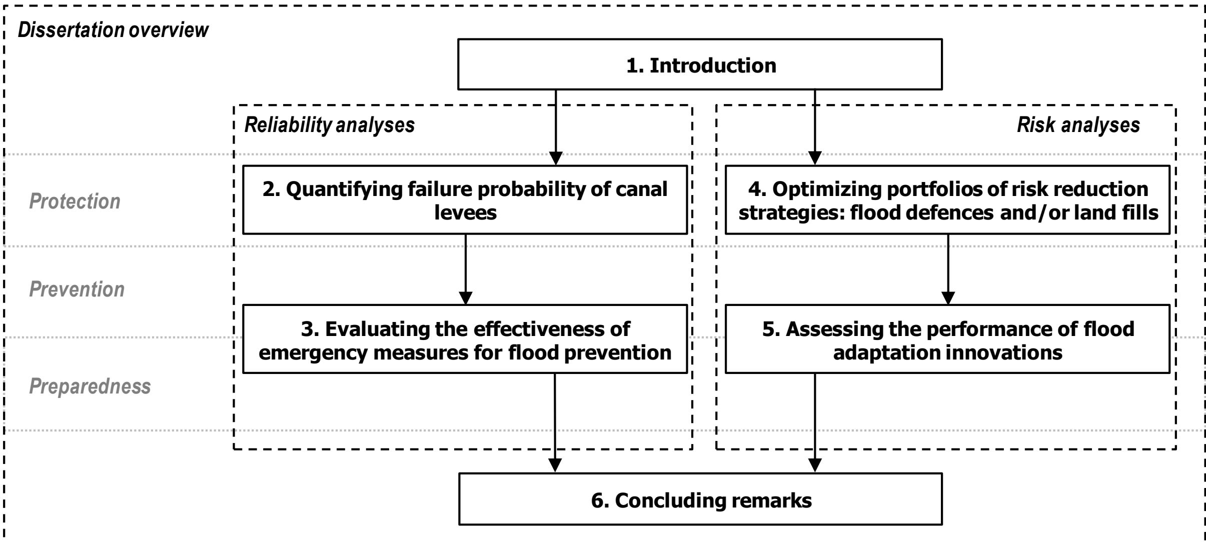

Figure 2: Dissertation overview, categorizing each chapter within a layer of a flood risk reduction system and the type of analysis used

Chapter 2 describes a method for quantifying the probability of failure of canal levees. Canal levees drain excess water out of polders to prevent pluvial flooding. With the introduction the full probabilistic approach, it is possible to take relevant aspects such as regulation of water levels, the probability of increased groundwater levels due to precipitation in to account and the uncertain presence of traffic loads in to account. Scenario analyses are used for this purpose. In addition, reliability updating is used to account for survived loads and determine the failure probability of the levee more accurately. A case study is included to illustrate how the method was used for a specific canal levee system in the northern part of the Netherlands.

Chapter 3 describes a method for evaluating the effectiveness of emergency measures for flood prevention. The method includes human reliability analyses for incorporating the uncertainty of human error when implementing emergency measures. Furthermore, logistical failure is included in the reliability analysis by comparing the time required for implementation with the available time before the flood hazard occurs. A case study is included that demonstrates the potential effectiveness of emergency measures along a river flood defence system in the eastern part of the Netherlands. Additionally, the cost-effectiveness of emergency measures is compared to the reinforcing the existing flood defence.

Chapter 4 investigates the drivers (i.e., flood defence and polder level) that determine flood risk in developments in flood prone areas and proposes a method to optimize elevation levels of land fills and flood defences. The method optimizes the elevation levels depending on the size of the area that requires protection and its land use. The model is used to discuss optimal elevation levels and preferred strategies for different land uses (and combinations). This will aid in better understanding of the cost and risk associated with both strategies and the drivers that influence these costs. Ultimately aiming to clarify why different risk reduction strategies may have been chosen and what may have driven these decisions.

In chapter 5, a framework is developed for the evaluation of the performance of ‘flood adaptation innovations’. Flood adaptation innovations are defined as solutions that have not been assessed in terms of risk reduction or solutions that have not yet been applied in practice. Examples include temporary flood barriers, green infrastructure and early flood warning systems. The framework requires innovators to evaluate their innovation with four performance indicators: effectiveness, durability, reliability and costs. The framework considers three testing phases before innovations are considered ready for market uptake: 1) a desk study, 2) laboratory testing and 3) operational testing. The performance of several practical examples of innovations is discussed to demonstrate how the framework can be applied in practice.

Finally, while each chapter concludes with their respective detailed findings and recommendations, the final chapter (6) discusses the main findings of this dissertation. The main findings are divided in those that address the specific techniques and methods developed within the risk-based approach and findings regarding the obtained results for specific applications. The main findings are followed by recommendations for further research, while also including recommendations addressed specifically at practitioners within flood risk management. This dissertation concludes with closing words.

2

Quantifying the failure probability of canal levees

Polders are protected from flooding by flood defence systems along main water bodies such as rivers, lakes or the sea. Inside polders, canal levees provide protection from drainage canals, which are used to prevent pluvial flooding by draining drain excess water from seepage and/or precipitation to the main water bodies outside of the polder. The water levels in these drainage canals are often regulated.

During the last decades, probabilistic approaches have been developed to quantify the probability of failure of flood defences along the main water bodies. This chapter proposes several extensions to this method to quantify the probability of failure of canal levees. These extensions include a method to account for i) water level regulation in canals, ii) the effect of maintenance dredging on the geohydrological response of the canal levee and iii) the inclusion of performance observations in the reliability analysis. By assessing the probability of failure of canal levees, decision makers are able to explore the relative benefit of risk mitigating measures for canal levees based on costs and risk reduction.

This chapter is based on the following publication in GeoRisk: Lendering, K.T., Schweckendiek, T., Kok, M. (2018). Quantifying the failure probability of a canal levee. Georisk: Assessment and Management of Risk for Engineered Systems and Geohazards. http://doi.org/10.1080/17499518.2018.1426865.

2.1 Introduction

Polders are often built in river deltas or low lying coastal areas to reclaim land. In the Netherlands, a large part of the country consists of polders, but polders are also found in Belgium, New Orleans, Sacramento or Bangkok. Polders typically lie below the surrounding water and are protected from flooding from the main water bodies by flood defences. These flood defences protect polders from the main hazards such as riverine or coastal flooding. Within these polders, large storage and drainage systems are made to drain excess water from the polders to the main water bodies. The drainage canals are aligned by canal levees that protect the surrounding polder from flooding from the inner water (inside the drainage and storage areas).

Traditionally, the strength of flood defences in the Netherlands is assessed using a semi-probabilistic approach (with safety factors) based on a statistically defined water level. In the last decades, full probabilistic approaches have been developed to assess the failure probability of flood defence systems accounting for the variability and uncertainty in both load and strength. The latter approach was used to quantify the probability of failure of flood defences along the main water bodies in the Netherlands, in the project “Flood Risk of the Netherlands” (Vrijling 2001; Jongejan et al. 2013)). The results of the project provided input for new safety standards for flood defences in the Netherlands (Rijkswaterstaat 2015), both in terms of cost-effectiveness of flood mitigation measures as well as considering risk to life (Jonkman 2005; Jonkman & Kok 2008a; Slijkhuis et al. 2001).

Canal levees were not taken in to account in the “Flood Risk of the Netherlands” project, even though there are several polders in the Netherlands with significant risk of flooding from the inner water bodies inside polders. For example, critical infrastructure such as the international airport of Schiphol and the HSL high speed rail line are both situated inside the Haarlemmermeerpolder, which is surrounded by a canal levee that aligns a large drainage canal system. Flooding from this canal system can result in significant (economic) flood damage. Furthermore, the dike breach at Wilnis in 2003 demonstrated that canal levees can breach at unexpected moments, in this case during a period of long drought in summer (Baars & Kempen 2009). Currently, the strength of canal levees is still assessed using a semi-probabilistic approach. The development of a full probabilistic approach can contribute to more effective flood risk management in areas at risk from flooding due to water bodies inside polders. This full probabilistic approach needs to take aspects specific to canal levees (and different from other flood defences) in to account, such as the regulation of water levels in canals and the occurrence of multiple loads on canal levees (e.g., water levels, rainfall and traffic loads).

This chapter proposes an extension of the approach to quantify the probability of failure of flood defences along the main water bodies to enable reliability analysis of canal levees. The application to the canal levee requires several additional features to account for i) regulation (and drainstop) of water levels in canals, ii) the possibility of (removal of) hydraulic resistance on the bottom of the canal due to maintenance dredging, iii) the uncertainty in traffic loads and iv) the uncertainty of the phreatic surface. The chapter is based on a more extensive technical report; more information on the discussed framework and case studies can be found in (Lendering et al. 2016). It is built up as follows. Section 2.2 describes the method proposed to quantify the probability of failure of a canal levee. In Section 2.3, we apply the method to a case study in the Netherlands. Finally, Section 2.4 contains the conclusions and recommendations.

2.2 Failure probability assessment

2.2.1 System description

Polders often lay below the main water bodies (e.g. a river, lake or sea) and are temporarily or permanently at risk of flooding. Water enters polders through groundwater flow, precipitation and/or inlet stations. Excess water is drained to the main water bodies through a drainage canal system. These drainage canals serve as (temporary) storage before the water is ultimately drained to the main water bodies. A schematized cross section of such a system is shown in Figure 3. Drainage canals are typically aligned by canal levees. Traditionally, these canal levees were constructed from locally available soil, often a mixture of clayey and peaty material. Seepage through the levees or bottom of the canal is limited due to the low conductivity of the materials used. Canal levees often are often also used for roads.

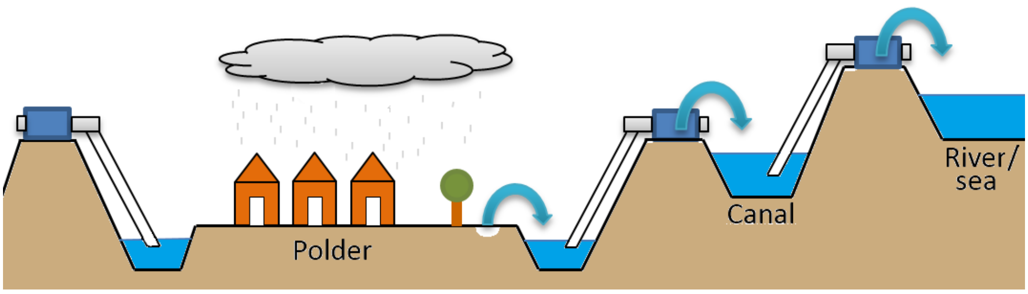

Figure 3: Typical cross section of a polder

The following subsection (2.2.2) discusses the general approach used to quantify the probability of failure of canal levee systems. The main loads on canal levees are discussed in Subsection 2.2.3, followed by a description of the considered failure mechanisms and how their probability is quantified in Subsection 2.2.4. Finally, Subsection 2.2.5 discusses a method to update the probability of failure using performance observations.

2.2.2 General approach

This chapter focusses on quantifying the probability of failure of canal levees. To this purpose, we will use the full probabilistic approaches applied in the VNK-2 project (Jongejan et al. 2013). An assessment of the consequences of flooding of canal levees and corresponding risk of flooding is beyond the scope of this chapter, but is treated in (K.T. Lendering et al. 2015).

The canal levee system is divided in sections with distinct, but homogeneous, strength properties, which allows independent modelling of sections in terms of strength. Failure is defined as breaching of the canal levee and occurs when the load [S] exceeds the resistance [R]. For example, a canal levee fails when the water level in the canal (i.e. the load) exceeds the retaining height of the levee (i.e. the resistance).

Limit state functions (Z) are defined for the dominant failure mechanisms of the considered canal levee. The limit state describes the condition beyond which the levee fails, in other words, the condition beyond which the resistance no longer exceeds the load. The general form of a limit state function is shown in Eq. 2.1, where the loads are described by the Solicitation (S) and the strength by the Resistance (R). The probability of the considered failure mechanism is quantified by the probability that the limit state function (Z) is smaller than zero (Eq. 2.2).

$$Z = \mathit{Resistance} - \mathit{Solicitation} \tag{2.1}$$

$$P_f = P(Z(x)<0) = P(Z(R,S)<0) \tag{2.2}$$

The cumulative distribution function (CDF) of the strength ($F_r(s)$) represents the conditional probability of failure mechanisms upon loading. Fragility curves illustrate the resulting conditional failure probability for the considered failure mechanism and load. These curves can be multidimensional depending on the number of loads considered (Vorogushyn et al. 2009). Through integration of the CDF of the strength ($F_r(s)$) over the probability density function (PDF) of the considered load ($f_s(s)$), we can determine the total yearly probability of the considered failure mechanism (Eq. 2.3).

$$P_f = \int_{r=-\infty}^{r=\infty} \int_{s=-\infty}^{s=\infty} f_{r,s}(r,s)\, dr\, ds = \int_{s=-\infty}^{s=\infty} f_s(s) \cdot F_r(s) ds \tag{2.3}$$

This equation is not solved analytically, because limit state functions of failure mechanisms are complex functions that can only be solved in a limited number of simple cases (Gouldby et al. 2008). Therefore, we propose to determine the CDF of the strength for a discretized set of load levels (E_j) using Level III (Monte Carlo simulations) and/or level II (first order approximation) probabilistic methods. The total failure probability is found after integrating the CDF of the strength over the PDF of the loads, taking dependence between the considered loads in to account. Depending on the considered loads, different load scenarios with corresponding probabilities are taken in to account using the law of total probability:

Observations of survived loads along these canal levees provide valuable information of the strength of the levee. These performance observations can be used to reduce uncertainties of the strength of the levee and therefore reduce the failure probability (Schweckendiek et al. 2014). After calculation of the probability of each failure mechanism, we will demonstrate how performance observations (survived loads) can be used to update the failure probabilities.

$$P_f = P(Z(x) < 0) = \sum_j P(f \mid E_j) \cdot P(E_j)$$

(2.4)

The probability of failure of the considered levee section is found by combination of the probability of each failure mechanism, taking dependence in to account. The upper and lower bounds of the failure probability are found by assuming mutually exclusive (upper bound) or complete dependence (lower bound) between failure mechanisms, see Eq. 2.5. In this equation, "i" represents each considered geotechnical failure mechanism and "n" represents the total amount of failure mechanisms considered.

$$\underset{i=1}{\overset{n}{MAX}}\, P_{f;i} \leq P_{f;sys} \leq \sum_{i=1}^{n} P_{f;i}$$

(2.5)

Based on experience obtained in the VNK2 project (Jongejan et al. 2013), we assume independence between failure mechanisms, allowing us to use Eq. 2.6 to calculate the probability of failure of the system. This assumption will be discussed further in the case study.

$$P_{f;sys} = 1 - \prod_{i=1}^{n} (1 - P_{f;i})$$

(2.6)

2.2.3 Main loads on canal levees

This section discusses the uncertainties of the main loads on the canal levees, being hydraulic (e.g. water levels) and traffic loads. Uncertainties are typically characterized by extreme value distributions. The main hydraulic loads consist of the water levels in the drainage canals and the phreatic surface in the canal levee (which influences the stability of the levee). Wave loads can generally be neglected, as the fetch on canals is typically insufficient to generate significant wind waves. Maintenance dredging can (unintentionally) increase the infiltration capacity of the bottom of the canal resulting in increased porewater pressure in the aquifer under the levee.

An overview of the main loads is shown in Figure 4. In our approach, the continuous probability density functions of these load variables are discretized in a predefined set of plausible load levels with corresponding probability density.

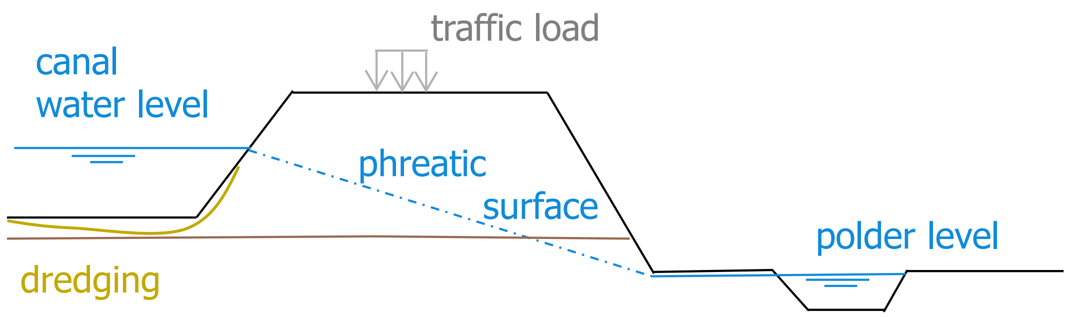

Figure 4: Cross section of a canal levee, illustrating the main loads acting on a canal levee

Water levels

Water levels in canals are influenced by inflow from the polder drainage stations, direct precipitation and drainage to the main water bodies. The water level in these drainage canals is regulated at a target level, which lies above the surrounding polders (see Figure 4). This target level is determined by a minimum required drainage or storage capacity in the canal or by other practical requirements, such as a minimum required navigation depth.

Besides the target level, a maximum target level is typically defined: the so-called "drainstop level", which aim to prevent extreme water loads on the canal levees. During heavy precipitation events, the pumping stations stop draining water from the polder to the drainage canal once the water level in the canal reaches the drainstop level, or maximum target level. The difference between the target level and the drainstop level is typically in the order of decimetres. Failure of the drainstop, i.e., failure of water level regulation (e.g. because local water authorities neglect, or forget, to turn off the pumping stations once the maximum target level is reached), can result in water levels exceeding the drainstop level.

A Generalized Pareto Distribution (GPD) is fitted to water level data to obtain the probability distribution (f_{GPD}) of the annual maximum water levels in the canal. In case of a perfectly working drainstop, the GDP would be truncated at the drainstop level and represented by ($f_{drainstop}$) in Figure 3. To account for water level regulation failures, a combined probability distribution (f(h)) of the canal water level is generated using the law of total probability, as defined in Eq. 2.7:

$$f(h) = f_{GPD} \qquad \textit{for } h < \textit{drainstop}$$

$$f(h) = P_{f;drainstop} \cdot f_{GPD} \qquad \textit{for } h > \textit{drainstop} \qquad (2.7)$$

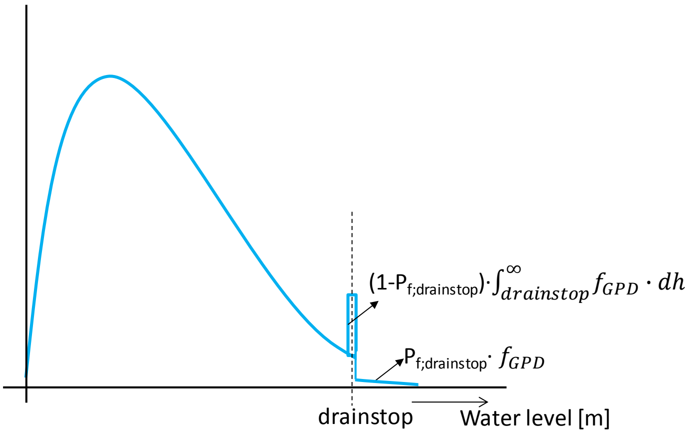

Figure 5: Probability distribution function of water levels, after accounting for regulation with a drainstop[1]

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1 This subsection was modified compared to the original publication to account for new insights regarding the probability distribution function of regulated water levels. Although the empirical model is slightly different from the original article, the results found in the case studies do not change signiciantly, because the exceedance probabilities of these water levels remains very small.

Here, ($P_{f;drainstop}$) is the probability of failure of the drainstop ($P_{f;drainstop}$) that can be estimated by the annual frequency of water level observations that exceeded the "drainstop level" (λ) using Eq. 2.8. The resulting annual exceedance frequency model of canal water levels is illustrated in Figure 6.

$$P_{f;drainstop} = 1 - e^{-\lambda t} \quad \textit{with } (t = 1 \text{ year}) \qquad (2.8)$$

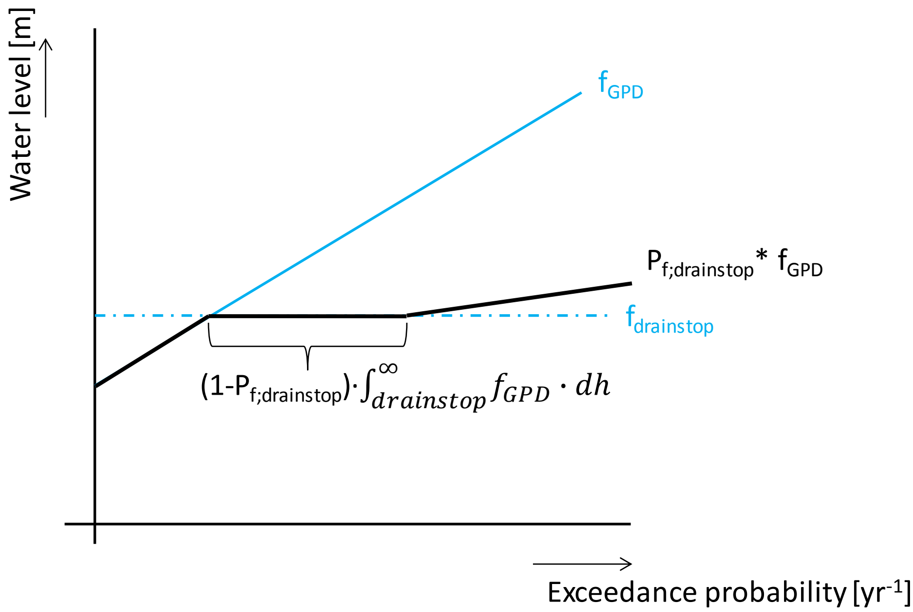

Figure 6: Annual exceedance frequency model of canal water levels, after accounting for regulation with a drainstop

An alternative to this empirical method is to determine the failure probability of the drainstop with a full reliability analysis taking human error in to account, an example of such an analysis for emergency measures is given in (Lendering et al. 2015; Kirwan 1996).

Phreatic surface

Without infiltration or evaporation, the phreatic surface inside the levee will reach a steady state: the canal-side boundary of the phreatic surface depends on the water level in the canal, while the land-side boundary of the phreatic surface depends on the water level in the polder. Rainfall (infiltration) and drought (evaporation) influence the saturation and, hence, the phreatic surface in time. The impact depends (among others) on the type of soil, the geometry of the levee and meteorological aspects (e.g., air moisture). Finally, the pore pressures induced by groundwater reduce the effective stresses in the soil and thereby the stability of the inner slope.

Groundwater flow models and/or monitoring of the groundwater table inside the canal levee can provide insight in the response of the phreatic surface to different forcing scenarios (e.g., heavy precipitation) with corresponding probability. However, research suggests that although different groundwater flow models can produce similar results, it remains difficult to reproduce observed groundwater levels (Esch

2012) and it is even more difficult to predict them. One main reason for the difficulty to model pore pressures by seepage analysis is the uncertainty in initial conditions in terms of the degree of saturation and the phreatic surface in daily conditions. Soil-atmosphere interaction in terms of precipitation and evaporation often results in groundwater trapped in the levee, at least in peat levees. At the same time, these processes are difficult to capture accurately in seepage analyses. Therefore, expert estimates based on experience with monitored or measured similar conditions are often as reliable as the results of seepage analyses. As for monitoring, the considered canal systems typically consist of tens or hundreds of kilometres of levee. Monitoring over the entire length of the system is typically not economically feasible. The expert judgement based approach should provide a reasonable first estimate in data-scarce conditions and the results can be perfectly used to target monitoring efforts to the risk hotspots.

Our specific, pragmatic proposal is to discretize the probability density function of the phreatic surface as a set of plausible levels dependent on two canal water levels: an average water level and an extreme water level (e.g., the drainstop level). A typical discretization contains three levels for the phreatic surface: low, average and high.

- A low level corresponds with a dry period, which may occur when the water levels in the canal are very low during a period of drought (no precipitation).
- An average level corresponds to the steady state situation with water levels at the target level.
- A high level corresponds to a situation where the levee is saturated, which may occur due to an extreme water level in the canal and/or during extreme precipitation.

With average canal water levels, the phreatic surface will likely be close to its steady state. Whereas with extreme water levels, which are the result of heavy precipitation, a high phreatic surface is most likely. The corresponding conditional probabilities can be estimated by, for example, members of water boards involved with the day-to-day maintenance of canal levees and often with knowledge of monitoring data from similar conditions.

Traffic loads

The combination of extreme hydraulic and traffic loads can be governing for the stability of a canal levee. Traffic loads are currently taken in to account deterministically as a static vertical load on top of the canal levee. We propose a probabilistic approach taking both the uncertain presence of the traffic load and the uncertainty of the magnitude of the traffic load in to account.

The presence of a traffic load on the canal levee depends on the considered canal levee (e.g., are there roads on top) and if flood fighting activities are expected during emergencies (e.g., will the local water board place sandbags on top of the levee to increase its height). To take this in to account, we will estimate the conditional probability of failure of the canal levee with ($P_{f;inst|tl}$) and without a traffic load ($P_{f;inst|\overline{tl}}$)) during all hydraulic loads, and use the law of total probability to account for the probability of traffic loads (P_{tl}).

$$P_{f;inst} = P_{f;inst|tl} \cdot P_{tl} + P_{f;inst|\overline{tl}} \cdot P_{\overline{tl}}$$
$$\text{with } P_{\overline{tl}} = 1 - P_{tl} \tag{2.9}$$

According to the guidelines for assessment of canal levees in the Netherlands (Stowa 2007), a static traffic load of 13,3 kN/m2 over a width of 2.5 meter in a plain-strain analysis needs to be taken in to account. This is the equivalent of a 12-meter-long, 40 ton vehicle. The effect of dynamic loads are assumed negligible in this study. For the purpose of modelling the traffic load probabilistically, water board employees were asked to provide estimates of the magnitude of average and extreme traffic loads, this is treated in more detail in Section 2.3.

2.2.4 Limit states of failure mechanisms

The probability of failure of canal levees is typically dominated by the probability of overflowing, instability and/or piping, whereas the contributions of other mechanisms such as instability of the revetment or wave overtopping are typically negligible. (no significant wave action). The limit state functions of the governing failure mechanisms are described in the following sections, followed by a description of how to quantify the probability of each mechanism. Fault tree analysis is used to combine the probability of each mechanism and quantify the failure probability of the considered canal levee section.

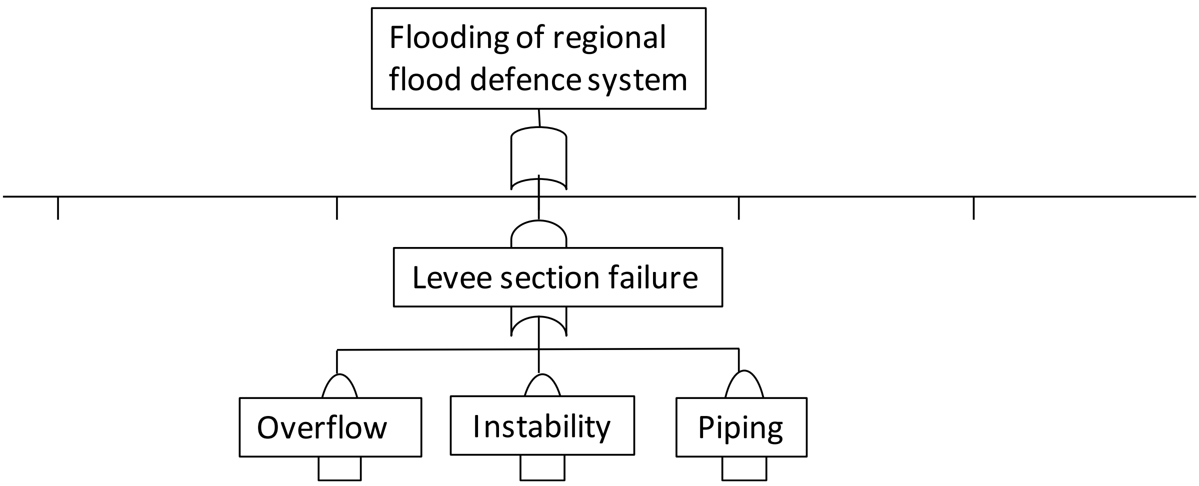

Figure 7: Simplified fault tree for the governing failure mechanisms of a canal levee section: overflow, instability and piping.

Overflow

Overflow occurs when the water levels in the canal (H_w) exceed the retaining height (crest level) of the levee (H_r), causing erosion of the inner slope. The limit state function

($Z_{overflow}$) considers a critical overflow height (Δhc), derived from a critical flow (q_c) that leads to erosion of the inner slope and ultimately breaching, see (Ciria 2014).

$$Z_{overflow} = H_r + \Delta h_c - H_w \text{ with: } \Delta h_c = \sqrt[3]{\frac{q_c^2}{0.36g}} \quad (2.10)$$

Note that the duration of overflow determines whether or not breaching of the considered levee will occur. In this chapter, we neglect the duration of overflow (which could amount up to several days) and assume failure to correspond with exceedance of the critical overflow height.

Piping

Piping occurs when the head difference over a levee causes internal erosion inside or through the body of a levee, which is the result of soil particles that are carried downstream by seepage flow. This can cause the formation of channels that undermine the levee and can ultimately cause breaching (Ciria 2014). The probability of piping depends on the head difference over the canal levee, which is the difference between the water level in the canal (H_w) and the polder level (H_i). The limit state function for piping considers a critical head difference (H_p), that is calculated with the updated Sellmeijer formula (Sellmeijer et al. 2011). Note that this formula is only applicable to loads with long durations, which is relevant for canal levees due to the regulation of water levels, but may not be applicable to other situations (e.g., river floods).

The water in the drainage canals is not always in direct contact with the aquifer below the canal levee; seepage to the surrounding polder is limited due to the low conductivity of the clayey / peat layers on the bottom of the canals. This so-called hydraulic resistance increases the resistance against piping. To account for hydraulic resistance, a variable (H_{ir}) is included in the limit state function of piping that effectively reduces the hydraulic head over the canal levee. The complete limit state function for piping is described by Eq. 2.11. The thickness of the blanker layer behind the levee is modelled by variable (D_0). The model parameter (m_b) takes in to account model uncertainty.

$$Z_p = m_b \cdot H_p - (H_w - 0.3 \cdot D_0 - H_i - H_{ir}) \quad (2.11)$$

The hydraulic resistance can be removed (temporarily) due to regular maintenance dredging of the canals, because dredging activities effectively remove the impermeable layers on the bottom of the canals. The probability of piping is estimated conditional on the hydraulic resistance, after which an estimate is made of the probability of removal of hydraulic resistance ($P_{\overline{ir}}$) based on the frequency of dredging activities and its impact (depth). The conditional probability of piping can be determined through Monte Carlo simulation or other reliability analysis techniques. The total probability of piping ($P_{f;p}$) is found after combining the conditional probability of piping given hydraulic resistance ($P_{f;p|ir}$) and removal of the hydraulic resistance

($P_{f;p|\overline{\mathrm{ir}}}$), taking in to account the probability of hydraulic resistance (P_{ir}) (using Eq. 2.12).

$$P_{f;\mathrm{p}} = P_{f;\mathrm{p|ir}} \cdot P_{\overline{ir}} + P_{f;p|\overline{\mathrm{ir}}} \cdot P_{ir}$$
$$\text{with } P_{\overline{\mathrm{ir}}} = 1 - P_{ir} \qquad (2.12)$$

Inner slope instability

Inner slope instability occurs when critical soil masses slide of the inner slope of the canal levee. The Bishop method (1955) is used to calculate the stability of the inner slope, for which the software D-Geo Stability (Deltares 2016) is used to determine the probability of failure. D-Geo Stability uses first order approximation methods (FORM) to determine the probability of inner slope failure conditional on a deterministic combination of the canal water level, the phreatic surface and the traffic load. Only slip circles that protrude the crest of the canal levee are taken in to account, as only these are considered to lead to breaching of the levee. The uncertainties in strength properties are based on the default values used in the VNK2 project (Jongejan et al. 2013).

The total probability of inner slope instability is determined after integration of the conditional failure probabilities over the joint probability distribution function of the considered loads. Assumptions regarding dependence between loads are discussed in the case study.

2.2.5 Using performance observations to update failure probabilities

Performance observations, such as the survival of extreme loads, can be used for reducing the uncertainty in a levees strength (Schweckendiek 2014). Along canal levees, the difference between average loads (e.g., the target water level) and extreme loads (e.g., the drainstop level) are typically limited to several decimetres, due to the regulation of water levels. Survived water levels near (or over) the drainstop level can provide valuable information. This information can be used to reduce strength uncertainties and update failure probabilities in a posterior analysis (also called Bayesian Updating).

Bayes' Rule forms the basis for updating probabilities with evidence of survived loads, see Eq. 2.13, where F is the failure event to be predicted (i.e. Z<0) and ε the observed event or evidence of the survived load (Schweckendiek 2014).

$$P(\mathrm{F} \mid \varepsilon) = \frac{P(\mathrm{Z(x)} < 0 \cap h(\mathrm{x}) < 0)}{P(\mathrm{h(x)} < 0)} \qquad (2.13)$$

The observation ε is described by the exceedance of an observational limit state expressed with an observational limit state function h(x), where (h) needs to be defined such negative values implies the observation to be true – in our case survival of the observed load. In the posterior analysis we assume the strength parameters to be time invariant and, hence, fully correlated between the survived and the predicted event More details regarding the steps used in the prior and subsequent posterior analysis are described in (Schweckendiek et al. 2014).

The effectiveness of a posterior analysis largely depends on the availability, accuracy and reliability of data of historically survived loads (ENW 2009). The potential influence of the posterior analyses on the failure probability increases when the survived loads or load effects (i.e. the survived water level) approach the extreme loads or load effects.

2.3 Case study in the "Heerhugowaard polder"

2.3.1 Case description

The approach described hitherto is applied to a system of canal levees surrounding a polder in the western part of the Netherlands. The polder is named the "Heerhugowaard" after the city that lies within, see Figure 8. The polder is surrounded by two large drainage canals that drain excess water from the polders to the North Sea: the "Schermer" and the "Verenigde Raaksmaats- en Niedorperkoggeboezem" (VRNK) canal. The 32-kilometer-long levee system is divided in six reaches for the purpose of flood risk analysis. Each reach consists of several sections as illustrated in Figure 8. In a flood risk analysis, the probability of failure of all sections within one reach would need to be combined to obtain the probability of flooding of the considered reach (i.e. a breach within the reach). The locations of four pumping stations along the levee system is also shown.

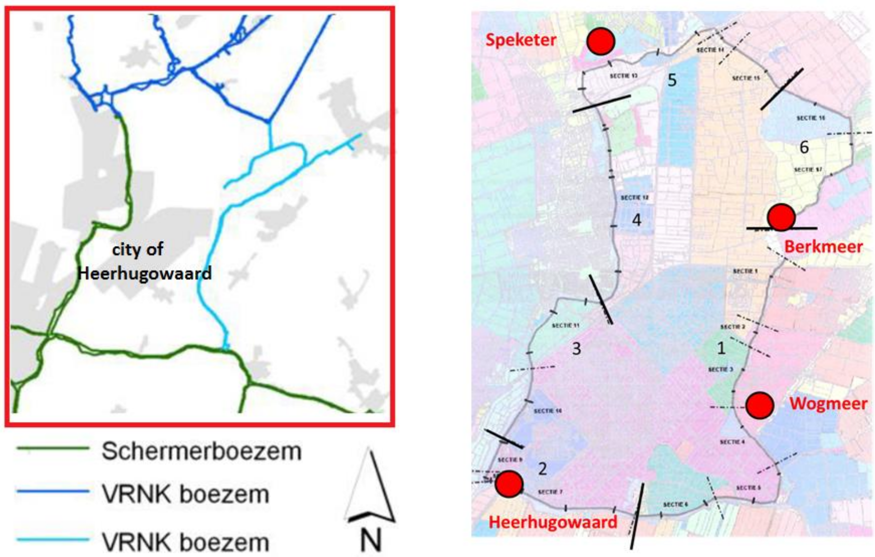

Figure 8: Overview of the Heerhugowaard polder surrounded by the Schermer and VRNK canals (left) and plan view of schematization of levee system including pumping stations (Lendering et al. 2015)

2.3.2 Load uncertainties

In our proposed approach, we estimated the probability density functions of the phreatic surface and traffic loads in a 'base case'. Additionally, we performed sensitivity analyses to investigate the sensitivity of the probability of instability to these estimates. The following subsections discuss the probability density functions of the water levels in the canal, the phreatic surface and traffic loads.

Water level

This subsection describes the probabilistic load models for the case study as described in general terms in section 2.2. The annual exceedance lines of the canal water level are determined using water level observations in the Schermer and VRNK canal. A GPD is fitted through independent water level peaks, which were determined using peaks-over-threshold (POT), after which the probabilities were corrected for the annual frequencies to obtain an annual exceedance line of water levels of the canal (Figure 9). Water levels are noted in meters relative to "Normaal Amsterdams Peil" or NAP.

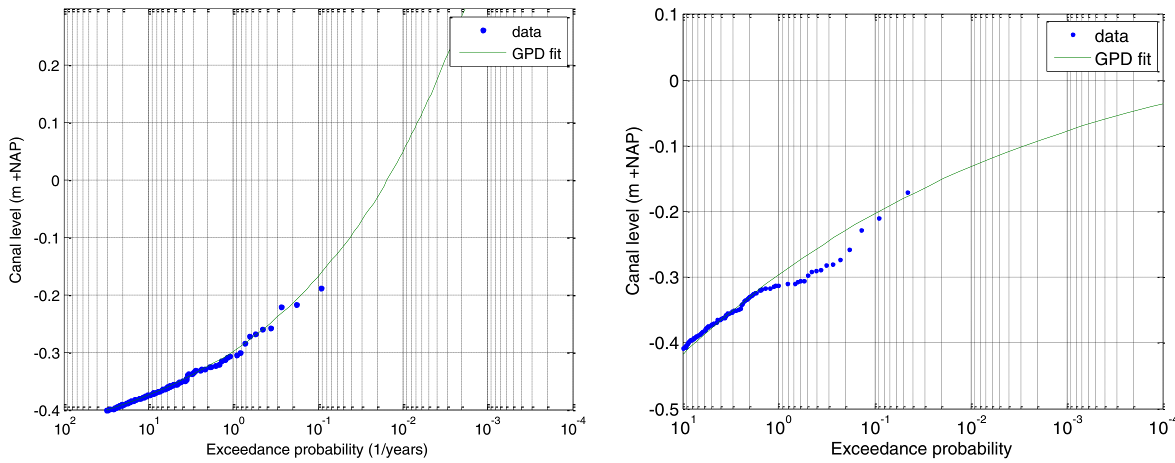

Figure 9: Generalized Pareto Distributions (GDP) fitted to independent water level peaks at Heerhugowaard (left) and Wogmeer (right) station

The drainstop level of each canal is shown in Table 2 and Figure 10. In the Netherlands, the water boards define the drainstop level by a target annual probability of exceedance of 1/100, which means that the annual probability of excess water in polders due to insufficient drainage to the canals is 1/100. However, the data of observed water levels shows that this target is not always met, meaning that the target level is reached more frequently. This could be because in practice, excess water in polders may occur more frequently due to extreme rainfall events (and volumes) that exceed the local drainage capacity. To prevent pluvial flooding in the polder, water is drained to the canals to levels above the drainstop level. In the considered case study, we used the observed frequency of exceedance of the drainstop level to obtain an estimate of the probability of exceedance of the drainstop level.

Location	Canal	Target level [m NAP]	Drainstop level [m NAP]	Frequency h>drainstop [yr-1]	Probability (h>drainstop) [yr-1]
Heerhugowaard	Schermer	-0.5	0	0	0.01 (1/100)
Wogmeer	VRNK	-0.6	-0.3	0.1303	0.1144 (1/9)

Table 2: Probability of water levels exceeding the drainstop level

The data set for the Heerhugowaard station was limited to 8 years, during which the drainstop level was never exceeded. Based on this data, no reliable estimate of the probability of exceedance could be made for this location. In absence of more data, for this canal we assume that the target annual probability is met. We therefore assume an annual probability of exceedance of 1/100 for the Schermer canal and 1/9 for the VRNK canal (based on the observations in that canal). On the VRNK canal, the data set for the Wogmeer station was limited to 22 years during which the drainstop level was exceeded several times. The combined annual exceedance frequency model of the water level of both canals, according to the approach described in Section 2.2, is shown in Figure 10.

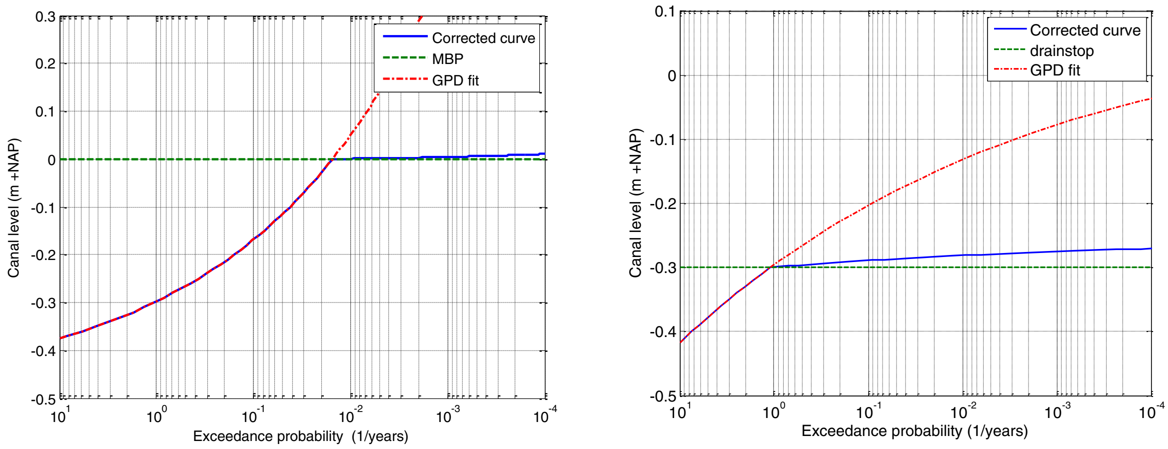

Figure 10: Annual exceedance frequency model of Heerhugowaard (left) and Wogmeer (right) pumping station

Phreatic surface

In Section 2.2, we proposed a discretized set of phreatic surfaces, conditional on the canal water level, that contains three levels: low, average and high. The correlation between both loads is determined by the amount and duration of precipitation within the canal system. The (simultaneous) response of the canal level and phreatic surface to precipitation depends on the size of the considered canal system and the properties of the considered canal levee (e.g. the infiltration capacity).

In absence of monitoring data, we assume a ‘base case’ with a positive correlation between the canal water level and the phreatic surface, as described in the following bullets:

- Given a low canal water level: it is most likely that the phreatic surface is low, representing a long dry period where canal water levels are low and levees dry out. However, the phreatic surface of dry levees is often raised arterially to prevent instability. Nevertheless, we assume that an average or high phreatic surface, given an average water level in the canal, has equal probability of 1/100. The remaining probability mass for the average phreatic surface level is 98/100.
- Given an average canal water level: it is most likely that the phreatic surface reaches a steady state between the canal side and polder side of the levee. We assume that a low or high phreatic surface, given an average water level in the canal, has equal probability of 1/100. This estimate is based on the probability of exceedance of the drainstop level in the canal. As explained, this scenario represents the likelihood of events that cause the canal water levels to reach the drainstop level. The remaining probability mass for the average phreatic surface level is 98/100.
- Given extreme canal water levels: with water levels in the canal reaching the drainstop level, it is most likely that the phreatic surface is high (and the levee saturated). Both an increase of the canal water level and the phreatic surface are caused by the same driver: precipitation. Therefore, we estimate the probability of a low and an average phreatic surface, given extreme canal water levels, to be 1/100. The remaining probability mass for the high phreatic surface level is 98/100.

The results are summarized in the conditional probability table displayed below:

		Phreatic Surface level		
		Low Phreatic surface	**Average Phreatic Surface**	**High Phreatic Surface**
Canal water level	Low water level	0.327	0.003	0.003
	Average water level	0.003	0.327	0.003
	High water level	0.003	0.003	0.327

Table 3: Conditional probability table of canal water level and phreatic surface

Traffic load

Regional water board employees responsible for the operation and maintenance of canal levees were asked to provide estimates of the 5_{th}, 50_{th} and 95_{th} quantiles of the statistical distribution of the traffic load, based on the weight of vehicles that are allowed on top of the levees. These estimates were used to generate a triangular probability density function. The results are presented in Figure 11. Compared to the deterministic value of the traffic load (13.3 kN/m), a higher expected value is found: 16.5 kN/m^2. Experts explained that the weight of the assumed design vehicle is underestimated (Stowa 2007), which is why they estimated the traffic load to be higher than the guidelines propose.

The regional water board employees all agreed that traffic loads can be expected during average situations, but had different views on whether or not traffic loads on canal levees can be expected during extreme situations (e.g., when the water level is at the drainstop level). Some argue that during extreme events, flood fighting will take place resulting in considerable traffic loads on the levee, while others argue that no traffic is allowed in this situation to avoid instability of the levee. To understand the impact of traffic loads on the probability of instability, in the 'base case' we calculate the conditional probability of failure of the levee and assume a probability of 1/2 for the presence of traffic loads (P_{tl}). In addition, with sensitivity analyses we will also estimate the probability of failure for situations with different probabilities of traffic loads.

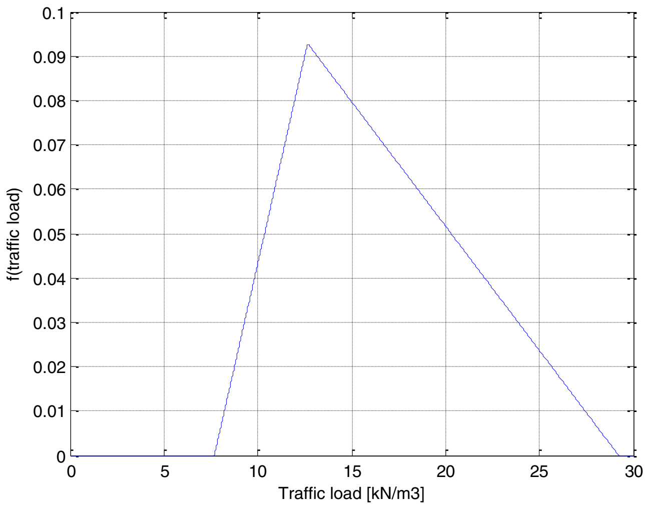

Figure 11: Triangular distribution of traffic load on canal levees in the Heerhugowaard

2.3.3 Application and results

This paragraph discusses the results of the failure probability assessment for Section 2.4 in reach 1 of the case study. This section is situated along the VRNK canal near the Wogmeer pumping station (Figure 8). Assumptions regarding case specific parameters used in the probabilistic calculations are included in each subsection. Details regarding piping input parameters are included in Appendix A.

Overflow

The resistance against overflow is determined by the retaining height (crest level) of the canal levee and the resistance to erosion of the inner slope. The input variables are shown in Table 4. The critical overflow amount depends on the overflow resistance of the inner slope cover layer. According to the guidelines (Stowa 2007), a maximum amount of 0.1 litres per meter per second is allowed. However, recent tests have proved that well developed grass cover layers can resist much more (EurOtop 2007). A deterministic critical overflow amount of 5 litre per meter per second is assumed, leading to a critical overflow height of 0.02 meter according to Eq. 2.9.

Variable	Parameter	Distribution	Mean	Standard deviation
Crest level	H_r	Normal	0.38 m	0.038 m
Critical flow	q_c	Deterministic	$5 \cdot 10^{-3}$ m^3/s	-
Critical overflow height	Δh_c	Deterministic	$2. \cdot 10^{-2}$ m	-

Table 4: Input variables overflow for section 4

The annual probability of overflow of the considered section is estimated smaller than 1/10,000. This low value is explained by the retaining height of the canal levee which lies well above canal water levels and corresponds to water levels with annual exceedance probabilities below 1/10,000 (see Figure 10). These low exceedance probabilities are explained partly by the low probability of exceedance of the drainstop level. The corresponding fragility curve is shown in Figure 12.

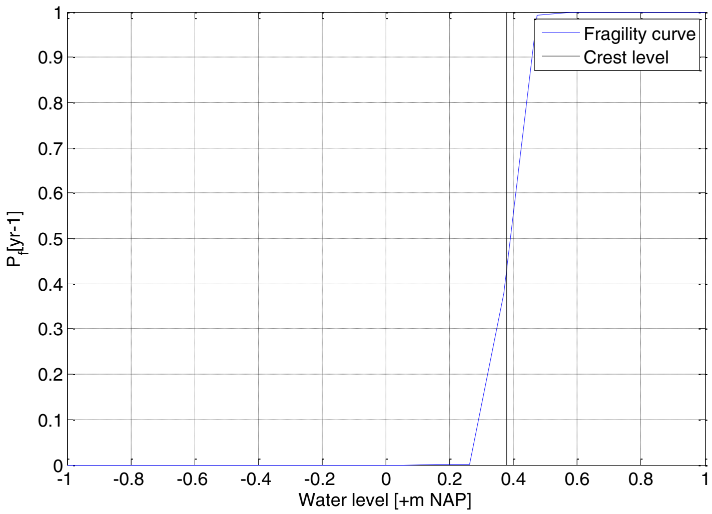

Figure 12: Overflow fragility curve (section 4)

Piping

The probability of piping depends on the amount of hydraulic resistance (H_{ir}) and the probability of (accidental) removal of hydraulic resistance due to dredging activities ($P_{\overline{ir}}$). The specific input parameters for section 4 are shown in Table 5, the remaining parameters used in the calculation are included in Appendix A.

Variable	**Parameter**	**Unit**	**Distribution**	**Mean (μ)**	**Coefficient of Variation (CV)**
Model parameter	m_b	-	LogNormal	1	0.12
Thickness of blanket layer	D_0	m	LogNormal	0.3	0.1
Polder level	H_i	m NAP	Normal	-3.9	0.1
Hydraulic resistance	H_{ir}	m	LogNormal	2.7	0.22

Table 5: Specific input variables piping for section 4

The probability of piping (Pf;p) conditional on the probability of removal of hydraulic resistance ($P_{\overline{\text{ir}}}$) is determined through Monte Carlo simulation, using the annual exceedance frequency model of Wogmeer station. Figure 13 illustrates the conditional probability of piping, which increases with increasing probability of removal of hydraulic resistance due to dredging ($P_{\overline{\text{ir}}} = 1 - P_{ir}$).

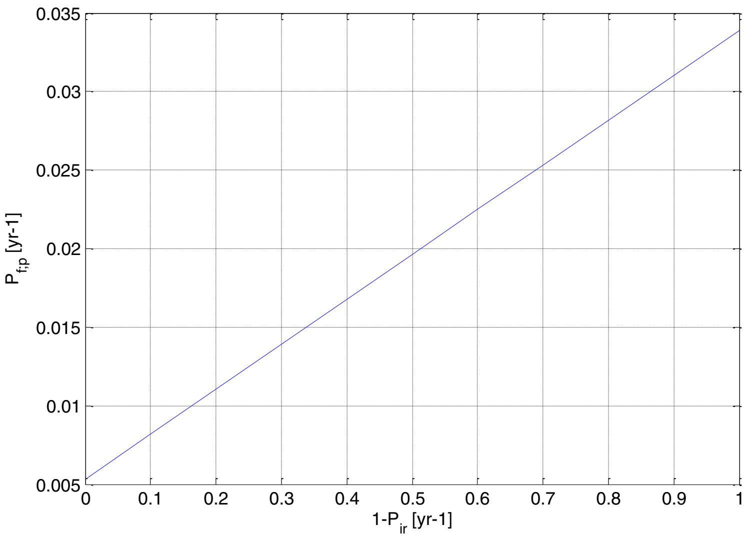

Figure 13: Piping failure probability conditional on probability of removal of hydraulic resistance (section 4)

The conditional annual probability of piping varies between a probability of 0.005 and 0.034, depending on the probability of removal of the hydraulic resistance. The VRNK canal is dredged every year. We assume a probability of (accidental) removal of the impermeable layer during maintenance dredging of 1/10 per dredging activity, which is common for activities subject to human error (Bea 2010). The resulting annual probability of removal of hydraulic resistance ($P_{\overline{\text{ir}}}$) is 1/10. The annual probability of piping for section 4 can now be calculated using Eq. 2.11 and amounts to 0.0082, or 1/120. The corresponding fragility curve for piping is shown in Figure 14.

The probability of piping is rather high given the fact that no signs of piping (e.g. heave or uplifting) have been observed along the canal levee during the last decades. Performance observations are used to further refine the probability of piping, using evidence of survived loads. The strength properties of piping lie in the geotechnical properties of the aquifer under the levee (e.g. the permeability and thickness of the aquifer and the seepage length of the levee). We assume the geotechnical properties of the survived event and the current situation to be perfectly correlated, because no large changes to (the geotechnical properties of) the levee have occurred in the period between the survived load until the current situation. A stepwise description of the method used is found in (Schweckendiek et al. 2014).

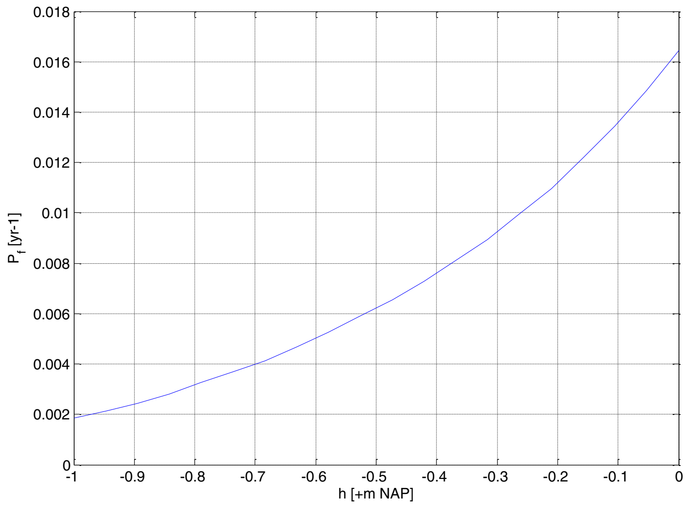

Figure 14: Piping fragility curve given an annual probability of removal of hydraulic resistance of 1/10 (section 4)

The highest observed water level in the VRNK canal is used in the posterior analysis. Due to lack of information of dredging during the observed water level, we assume that the impermeable layer on the bottom of the canal was present during the survived load. This load case is defined as load case h1, see Table 6.

	Survived water level (H_s)	**Mean of hydraulic resistance (H_{ir})**
Load case h1	-0.17 m NAP	2.7 m
Load case h2	-0.17 m NAP	0 m

Table 6: Properties of load cases for posterior analysis of piping

The a-posteriori probability of piping given survival of load case h1 is calculated with Eq. 2.13. The survival of load case h1 results in a reduction of the annual probability of piping to a range of $<10^{-5}$ and 0.026, as can be seen in Figure 15. Assuming an annual probability of removal of the hydraulic resistance due to dredging of 1/10, we find an annual probability of piping of 0.0027.

Suppose that the impermeable layers on the bottom of the canal were removed right before the survived load occurred, resulting in the absence of hydraulic resistance. This load case is defined as load case h2, see Table 6. The hydraulic head during this hypothetical load case is significantly higher than the average hydraulic head over the levee (in the order of several meters), due to the absence of hydraulic resistance. The conditional annual probability of piping given this hypothetical load case ($P_{f;posterior|h2}$) is smaller than 10^{-5} as shown in Figure 15. The resulting probability of piping is more realistic for the considered dike section considering the absence of signs of piping. Especially considering that it is highly probable that during the last decades, the intrusion resistance was removed given the assumed annual probability of 1/10. However, we do not know if this scenario occurred together with an average or extreme water level. Therefore, this hypothetical load case demonstrates the ability of using test loading to further refine failure probabilities.

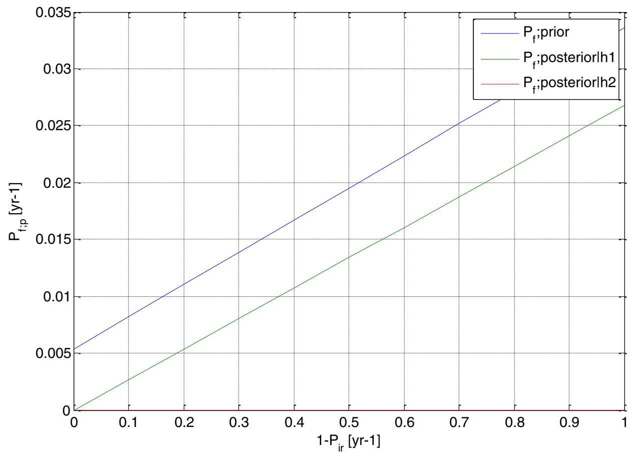

Figure 15: The a-priori conditional probability of piping ($P_{f;prior}$), a-posteriori probability given load case h1 ($P_{f;posterior|h1}$) and load case h2 ($P_{f;posterior|h2}$)

Ultimately, we conclude (conservatively) that the probability of piping is determined by the a-posteriori probability given load case h1 and an annual probability of removal of hydraulic resistance of 1/10. This results in an annual probability for piping of ($0.026 \cdot 0.1 + 10^{-5} \cdot 0.9$) = 0.0026, or 1/384. Figure 16 illustrates the fragility curve of the a-priori probability of piping, the a-posteriori probability given load case h1 and load case h2.

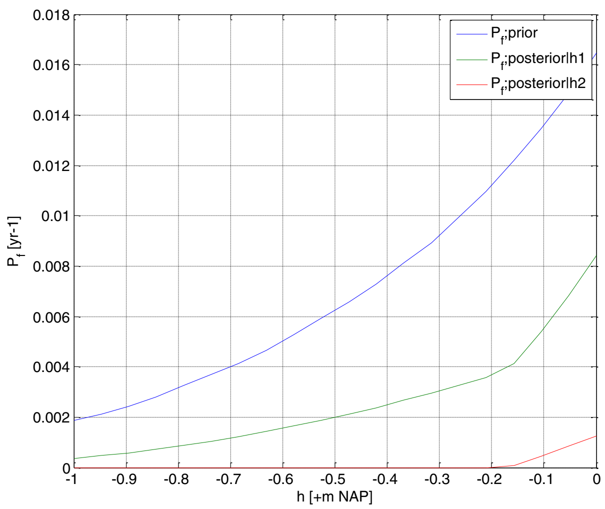

Figure 16: Piping fragility curves for the a-priori failure probability, the a-posteriori probability given load case h1 and load case h2. Each load case considers a annual probability of removal of hydraulic resistance of 1/10

The probability of piping is dominated by the conditional annual probability of piping given no hydraulic resistance (0.026). This study assumed an annual probability of removal of the hydraulic resistance of 1/10. Reduction of this probability will reduce the probability of piping. Furthermore, load case h2 demonstrated that the probability of piping can be further reduced using a test load on the canal levee, consisting of the removal of the impermeable layer on the bottom of the canal and raising the water level in the canal.

Inner slope instability

The stability of the inner slope depends on three loads: i) the water levels in the canal, ii) the phreatic surface in the levee and iii) the traffic load on top of the levee. Cross sections of the schematized canal levee in D-Geo stability are shown in Figure 17. The illustrated schematization shows a canal levee with a phreatic surface representative for a steady state situation, conditional on the canal water level. The conditional probability of inner slope instability is calculated with D-Geo Stability for several deterministic combinations of these loads, the results are included in Table 7.

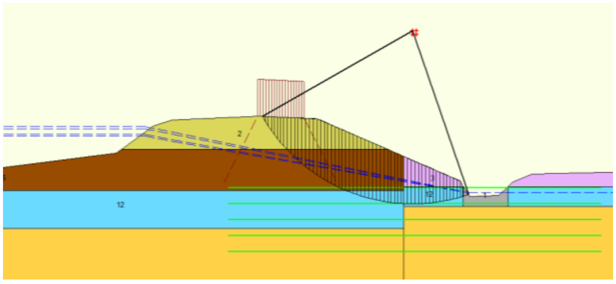

Figure 17: Cross section of schematization of section 4 in D-Geo Stability showing the canal water level, the phreatic surface and traffic loads. Also displayed is a slip circle that would lead to breaching.

Traffic load = 0kN/m^2			
Water level (m NAP)	Low Phreatic Surface	Average Phreatic Surface	High Phreatic Surface
-0.67 m NAP	$1.3 \cdot 10^{-8}$	$2.5 \cdot 10^{-8}$	0.008
-0.59 m NAP	$1.3 \cdot 10^{-8}$	$2.5 \cdot 10^{-8}$	0.009
-0.30 m NAP	$1.3 \cdot 10^{-8}$	$2.5 \cdot 10^{-8}$	0.015
-0.17 m NAP	$1.3 \cdot 10^{-8}$	$2.5 \cdot 10^{-8}$	0.019
Traffic load = 5kN/m^2			
Water level (m NAP)	Low Phreatic Surface	Average Phreatic Surface	High Phreatic Surface
-0.67 m NAP	$2.4 \cdot 10^{-8}$	$4.4 \cdot 10^{-8}$	0.011
-0.59 m NAP	$2.4 \cdot 10^{-8}$	$4.4 \cdot 10^{-8}$	0.019
-0.30 m NAP	$2.4 \cdot 10^{-8}$	$4.4 \cdot 10^{-8}$	0.024
-0.17 m NAP	$2.4 \cdot 10^{-8}$	$4.4 \cdot 10^{-8}$	0.024
Traffic load = 13kN/m^2			
Water level (m NAP)	Low Phreatic Surface	Average Phreatic Surface	High Phreatic Surface
-0.67 m NAP	$1.2 \cdot 10^{-7}$	$2.1 \cdot 10^{-7}$	0.018
-0.59 m NAP	$1.2 \cdot 10^{-7}$	$2.1 \cdot 10^{-7}$	0.022
-0.30 m NAP	$1.2 \cdot 10^{-7}$	$2.1 \cdot 10^{-7}$	0.038
-0.17 m NAP	$1.2 \cdot 10^{-7}$	$2.1 \cdot 10^{-7}$	0.048
Traffic load = 30kN/m^2			
Water level (m NAP)	Low Phreatic Surface	Average Phreatic Surface	High Phreatic Surface
-0.67 m NAP	$5.0 \cdot 10^{-7}$	$8.8 \cdot 10^{-7}$	0.033
-0.59 m NAP	$5.0 \cdot 10^{-7}$	$8.8 \cdot 10^{-7}$	0.039
-0.30 m NAP	$5.0 \cdot 10^{-7}$	$8.8 \cdot 10^{-7}$	0.068
-0.17 m NAP	$5.0 \cdot 10^{-7}$	$8.8 \cdot 10^{-7}$	0.087

Table 7: Annual probability of instability for combinations of water level and phreatic surface given a deterministic traffic load (section 4)

The results show that increasing water levels have a negligible effect on the probability of failure for a low and an average phreatic surface (specifically within the estimated slip circles of the inner slope, see Figure 17). In contrast, for a high phreatic surface, increasing water levels in the canal do result in increasing probabilities of failure. This is also visible in the fragility curves illustrated in Figure 19. The figure illustrates the probability of failure conditional on each load after integration over the joint probability distribution function of the remaining loads. For example, the fragility curve to the left illustrates the failure probability conditional on the water levels after integration over the conditional probabilities of the phreatic and the traffic load (illustrated in Figure 11). Specifically, the scenario with extreme water levels (near or over the drainstop level), combined with a saturated levee due to a high phreatic surface, result in the highest conditional probability of failure.

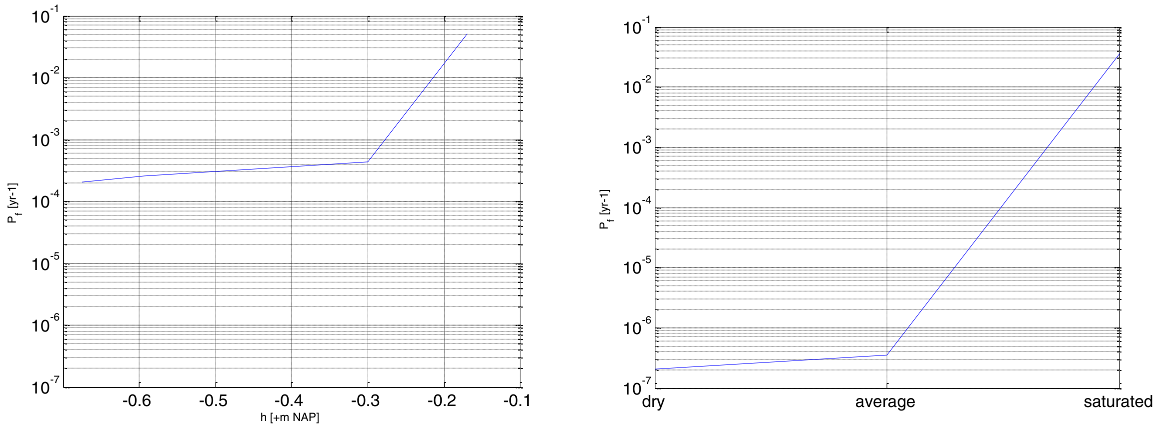

Figure 18: Fragility curves for inner slope instability conditional on the canal water level (left) and the phreatic surface (right)

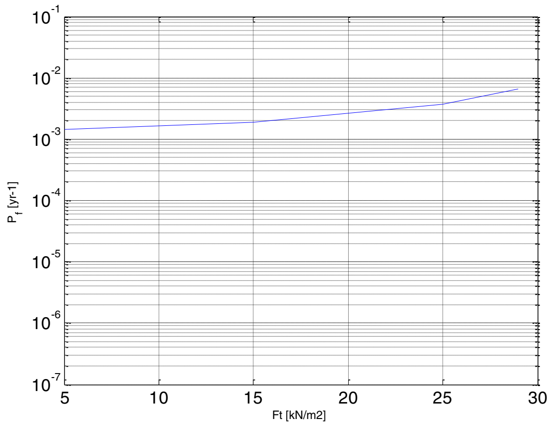

Figure 19: Fragility curves for inner slope instability conditional on the traffic load

Given the estimates of the water board employees regarding traffic loads (see Section 2.2), we assume that traffic load on top of the levee is independent of the occurring hydraulic loads. The conditional failure probabilities are subsequently integrated over the probability distribution function of each load as defined in the 'base case' and summed. The results are shown in Table 8.

Following the table is a graph that illustrates the sensitivity of the probability of instability on the probability of traffic loads (Figure 20).

	Parameter	Value
Probability of inner slope instability without traffic loads	$P_{f;inst\|\overline{tl}}$	$1.5 \cdot 10^{-3}$
Probability of inner slope instability with traffic loads	$P_{f;inst\|tl}$	$4.0 \cdot 10^{-3}$

Table 8: Annual probability of inner slope instability conditional on the presence of traffic loads, for the 'base case'

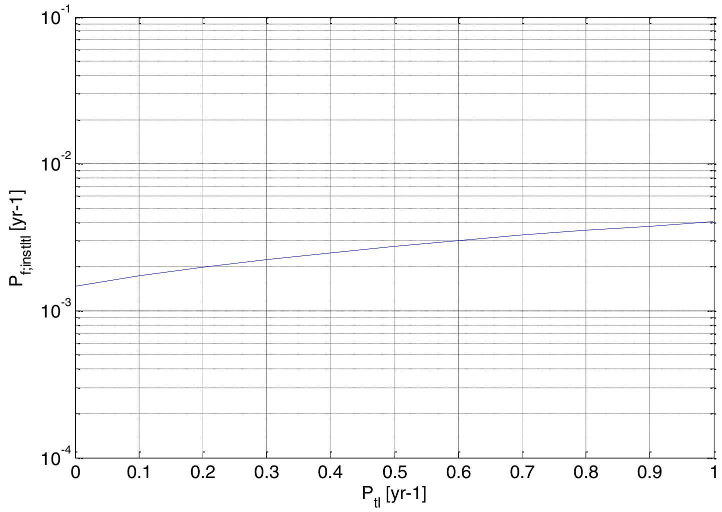

Figure 20: Annual probability of inner slope instability conditional on the presence of traffic loads, for the 'base case'

Sensitivity analyses were also performed to study the impact of the estimates of the conditional probability table of the phreatic surface and the probability of traffic loads. In the base case, the probability of a phreatic surface other than the level most likely to occur given a certain water level was estimated at 1/100. This value was varied between 1/10 and 1/1,000 to study the sensitivity of the probability of instability to this value. The results are shown in Table 9. A reduction of the probability of 'other' phreatic surfaces does not to have a large effect. An increase with an order of magnitude results in a larger difference, but probabilities remain within the same order of magnitude. Thus, the sensitivity to these assumptions is not very large.

	Parameter	Value
Probability of inner slope instability, probability of 'other' phreatic surface of 1/100 (base case)	$P_{f;inst}$	$2.7 \cdot 10^{-3}$
Probability of inner slope instability, probability of 'other' phreatic surface of 1/1000	$P_{f;inst}$	$2.6 \cdot 10^{-3}$
Probability of inner slope instability, probability of 'other' phreatic surface of 1/10	$P_{f;inst}$	$4.3 \cdot 10^{-3}$

Table 9: Sensitivity analyses showing the impact of varying conditional probabilities of the phreatic surface. Here, the 'other' phreatic surfaces are defined as the levels that are not most likely to occur given a certain water level.

Performance observations can be used to further refine the probability of instability, using evidence of survived loads (Schweckendiek & Vrouwenvelder 2013). Initiation of instability failure can be observed by movements of the levee in downstream direction causing cracks in the levee. Such signs of failure/ performance can be incorporated in a Bayesian Analysis, similar to what was done for piping. The formation of cracks could be considered as an initiated slope failure within the analysis used to update the estimated prior failure probabilities.

The strength properties of the instability failure mechanism are determined by both the geometrical and geotechnical properties of the levee. Similar to the piping mechanism, we can assume that these properties are highly correlated between the survived event and the predicted (future) event, if we can exclude the possibility of large changes over time. However, posterior analyses only have a large impact on the probability of failure if the water level in the canal is dominant over other loads. The results of this study demonstrated that the phreatic surface, influenced by precipitation, is dominant. Therefore, performing a similar posterior analysis for instability as done for piping may not have a large impact on the probability of failure, because insight is required in the level of the phreatic surface at the time of the survived load.

2.3.4 Probability of failure

The annual probability of each failure mechanism for the considered dike section is summarized in Table 10. Failure probabilities are combined assuming independence between the failure mechanisms, to obtain an estimate of the annual probability of failure of the considered dike section, according to Eq. 2.9. When comparing the probabilities of the failure mechanisms, we conclude that the geotechnical failure mechanisms (i.e., piping and instability) are dominant, with failure probabilities of piping and instability having the same order of magnitude. The assumption of independence between failure mechanisms provides an upper bound of the actual failure probability. The lower bound, given complete dependence between respective failure mechanisms, is the largest annual failure probability, which is 1/370 for instability.

Failure mechanism	**Parameter**	**Value**
Overflow	$P_{f;o}$	< 0.000001 (1/100,000)
Piping (with performance observations)	$P_{f;p}$	0.0026 (1/ 384)
Instability	$P_{f;inst}$	0.0027 (1/ 370)
Upper bound probability of failure	P_f	0.005 (1/200)

Table 10: Annual probabilities of failure for section 4 of case study

This chapter has demonstrated the assessment of probabilities of failure of one dike section. To assess probabilities of failure on system level we need to consider the length-effect within homogeneous sections as well as the system reliability when combining the various sections in the system, which essentially work as a parallel system. Methods that can be used for this purpose are explained in (Kanning 2012).

2.4 Concluding remarks

This chapter proposes an approach to quantify the probability of failure of a canal levee, based on the probabilistic methods developed for flood defences along the main water bodies. In our approach, the continuous probability density functions of several load variables are discretized in a predefined set of plausible load levels with corresponding probability density. The total law of probability was used to account for i) regulation (and drainstop) of water levels in canals, ii) maintenance dredging and its influence on the hydraulic resistance of the canal, iii) the uncertain presence of traffic loads and iv) the uncertainty of the phreatic surface. In addition, reliability updating is used to demonstrate the impact of incorporating performance observations for the piping failure mechanism.

The proposed approach was applied to a case study of a canal levee system in the Netherlands. The probability of three failure mechanisms was determined: overflow, piping and instability. The probability of overflow was dominated by the probability of drainstop failure. For piping, the probability of failure was largely influenced by the (probability of) hydraulic resistance of the bottom layer of the canal. A posterior analysis demonstrated the ability to reduce the probability of piping using performance observations. The posterior analysis opens opportunities for testing the piping resistance of a canal levee under different combinations of loads.

The probability of instability of the inner slope was dominated by the uncertainty in the phreatic surface and traffic loads. We conclude that the probability of failure of the considered dike section is governed by the probability of piping and instability. The probability of overflow is negligible.

Based on the results of this chapter, we recommend further investigating dependencies between canal water levels and the phreatic surface, also taking into account system size and capacity, and the potential of incorporating performance observations in the quantification of the probability of instability of the inner slope. Further studies could use test loadings (e.g., by artificially increasing the phreatic surface and/or canal level) on existing levee sections to assess their performance under design conditions.

Overall, we conclude that the proposed approach can be used to quantify the probability of failure of canal levees. By doing so, the approach contributes to improving flood risk management of canal levee systems by providing input for risk assessments of canal levee systems. With these methods, it becomes possible to evaluate and prioritize different flood risk reduction measures (e.g., levee reinforcement or increasing drainage capacity) in terms of their costs and benefits (or risk reduction).

3

Evaluating the effectiveness of emergency measures for flood prevention

Emergency measures are defined here as temporary measures implemented during a (threatening) flood to reinforce, or repair, damages in flood defences and prevent breaching. Examples of emergency measures are placing sand bags on top of flood defences to gain more height or constructing a soil berm against flood defences for more horizontal stability. Even though decision makers often apply emergency measures during flood events, insight in their effectiveness in terms of risk reduction is limited.

Within a full probabilistic approach, aspects such as organizational failure, operator error and logistical failure can be included in the overall assessment of reliability and risk. By doing so, this chapter aims to quantify the reliability of emergency measures and to assess their effectiveness in terms of risk reduction. Ultimately, this chapter compares the costs and effectiveness of emergency measures with permanent reinforcements of flood defences to aid decision makers in deciding between the implementation of one or the other.

This chapter is based on the following publication in Journal of Flood Risk Management: Lendering, K. T., Jonkman, S. N., Kok, M. (2016). Effectiveness of emergency measures for flood prevention. Journal of Flood Risk Management, 9(4). http://doi.org/10.1111/jfr3.12185

3.1 Introduction

During flood events, local authorities, civilians and armies often work together to place tens of thousands of sand bags, attempting to prevent large breaches in flood defences. These events show that emergency measures, such as sand bags and big bags, are often used during threatening floods to reinforce flood defences or protect critical infrastructure from flooding. Despite these attempts, dike breaches often occur, causing large floods in the otherwise protected areas (Ellenrieder & Maier 2014).

Emergency measures are defined here as temporary measures implemented during a (threatening) flood to reinforce, or repair, weak spots in flood defences. Weak spots are damaged sections of the flood defence, where a breach can occur during the expected river flood if no measures are taken. The required emergency measure depends on the type of flood defence, the extent of damage and the length over which the flood defence is damaged.

The reliability of permanent flood defences, such as dikes (water-retaining soil structures), is analysed extensively in Jongejan et al. (2013) and Vrijling (2001). The influence of human and organizational systems on the reliability of flood defences has been studied in the context of the closure of complex storm surge barriers (Vrancken et al. 2008). However, the influence of emergency measures is commonly omitted in these analyses, because there is limited insight into their reliability and effect on flood risk. According to Leeuw et al. (2012), the reliability of emergency measures is largely determined by human, organizational and logistic aspects. A similar conclusion is drawn in Corn & Inkabi (2013), who determine the effect of human intervention on the reliability of flood defences using Quality-Management Assessment System (QMAS), developed by Bea (1998). While this chapter took in to account human reliability, it failed to account for the possibility of having insufficient time to implement emergency measures. Furthermore, the structural reliability of the emergency measures was not analysed and integrated with the reliability of the permanent flood defence system.

The objective of this chapter is to develop a method which determines the reliability of a permanent flood defence system including emergency measures. The reliability of emergency measures is quantified through an extensive reliability analysis, which not only takes structural reliability of the emergency measure in to account but also includes the reliability of the people and logistics during implementation of the emergency measure. Justification of emergency measures as a flood risk reduction strategy is investigated by means of a cost benefit analysis, comparing this option to the ‘do nothing’ situation and traditional dike reinforcements.

We focus on emergency measures which aim to prevent initiation of breaching of river dikes. Emergency measures which aim to stop breach growth and/or to close breaches are beyond the scope of this chapter; these are investigated in van Gerven (2004) and Joore (2004). The main failure modes of river dikes are overflow, piping, inner slope instability and outer slope erosion, according to VNK (2005). We considered overflow and piping, because these are the dominant failure modes of dikes in the case study area and in the Netherlands (VNK 2005). This chapter is based on a more extensive technical report (Lendering et al. 2013) in which more information on the discussed method and case study can be found .

The chapter is structured as follows. Section 3.2 describes the method developed to determine the reliability of emergency measures. In Section 3.3, we apply the method to a case study of a specific flood defence system. The results are discussed as well as a sensitivity analysis. Section 3.4 describes an approach to determine the cost-effectiveness of emergency measures as a flood risk reduction strategy. It is applied to the case study as well. Conclusions regarding the method and case study results are given in Section 3.5, followed by recommendations for further research.

3.2 Method for the reliability analysis of emergency measures

3.2.1 General approach

An analysis of the reliability of a system can be performed with different methods (e.g., event and fault tree analysis, Bayesian Networks and N2 diagrams). Event tree analysis is used in our approach, because event trees provide a deductive, top-down method to relate different possible outcomes to a single initiating event. In this case, the initiating event being a flood hazard such as a river flood or storm surge. After occurrence of such an initiating event, different events need to be completed successively in order for a combination of emergency measures and flood successfully defences to protect an area from flooding.

The general approach of the method is shown in the event tree in figure 1. It assumes the occurrence of a flood event, in this case a high discharge in a river, and the implementation of emergency measures. The first event in the figure models the reliability of the emergency measures, whereas the second event models the reliability of the (existing) flood defence. In this chapter we define success as the compliment of failure. The reliability is defined as the probability of success, which is the compliment of the failure probability, see Eq. 3.1. The effectiveness of the emergency measure is defined as the extent to which the reliability of the flood defence system is increased by implementation of emergency measures.

$$\textit{Reliability} = 1 - \textit{Failure Probability}\ (P_f) \qquad (3.1)$$

Flooding will occur if the flood defence system fails forming a breach in the flood defence. This will happen when the emergency measure fails *and* the flood defence fails (top branch in Figure 21) or when the flood defence fails *in spite* of a correct functioning emergency measure (bottom branch in Figure 21). Thus, even when emergency measures are successfully applied, the flood defence can still fail; for example, when a dike covered with sandbags overflows.

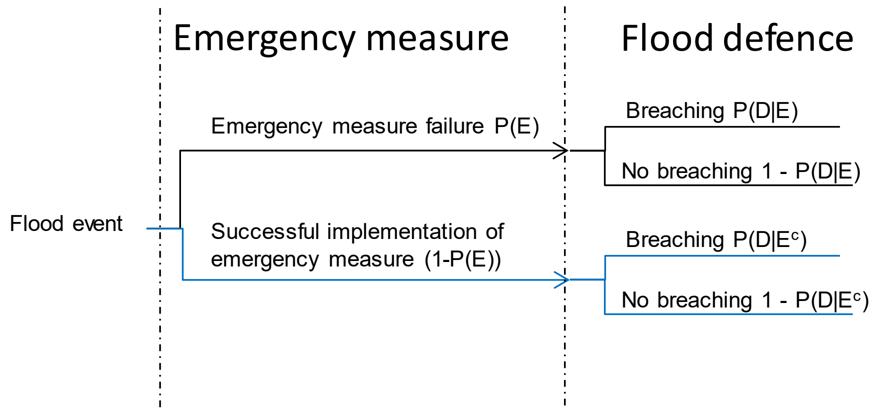

Figure 21: General event tree to determine the reliability of flood defence with emergency measures

The failure probability of a flood defence system while taking in to account implementation of emergency measures is determined in four steps:

1) Determine the failure probability of the emergency measures [P(E)] (Subsection 3.2.2);
2) Determine the failure probability of the flood defence system without emergency measures, which is equal to the failure probability of the flood defence system given failure of emergency measures [P(D|E)] (Subsection 3.2.3);
3) Determine the failure probability of the flood defence system given successful implementation of emergency measures [$P(D|E^c)$] (Subsection 3.2.3);
4) Determine the combined failure probability of the flood defence system taking in to account implementation of emergency measures [P_f], by solving Eq. 3.2 (Subsection 3.2.4).

$$P_f = P(E) * P(D|E) + P(E_c) * P(D|E_c)$$
$$\text{with } E_c = 1- P(E) \qquad (3.2)$$

For example, placing sand bags on top of a dike will increase the retaining height of the dike, therefore increasing the strength of the flood defence against overflow. The combined overflow probability of the dike including sand bags is determined by combining the failure probability of implementation of the sand bags with the failure probability of the dike with a higher retaining height. While this method assumes a binary approach to combine the reliability of emergency measures with the reliability of the flood defence system for specific failure modes, it is recommended to assess the effect of emergency measures on all failure modes of the considered flood defence to assure that the overall failure probability is reduced.

3.2.2 Reliability of emergency measures

General method

This subsection proposes a method to determine the failure probability of emergency measures, which is modelled by the first event in Figure 21. The implementation of emergency measures is divided into three sub events: 'Detection', 'Placement' and 'Construction'; a similar distinction is made by Corn and Inkabi (2013). These sub events need to be completed successfully in order for the emergency measure to function. They are modelled in an event tree forming a series system, see Figure 22. This event tree represents the first event of the general event tree (Figure 21). A description of each sub event is given below.

1. 'Detection', during this sub event, the upcoming high water is monitored and the flood defences are inspected to find possible weak spots where emergency measures are required. Failure can occur due to i) human error or ii) insufficient time to inspect all flood defences.
2. 'Placement': during this sub event, the required emergency measures are implemented. Failure can occur due to i) human error or ii) insufficient time to finish placing all emergency measures.
3. 'Construction': during this sub event, the emergency measure needs to function correctly to prevent failure to the flood defence. Failure can occur due to structural failure of the emergency measure (e.g. due to instability).

Both 'Detection' and 'Placement' are subject to two different types of failure modes: i) human error and ii) failure due to insufficient time. 'Construction' is subject to one failure mode: structural failure of the emergency measure. Failure of one of the sub events leads to failure of the emergency measure. Flooding will however only occur when the flood defence breaches, which may occur if the emergency measure fails, but also if the emergency measure is successful and the flood defence still fails.

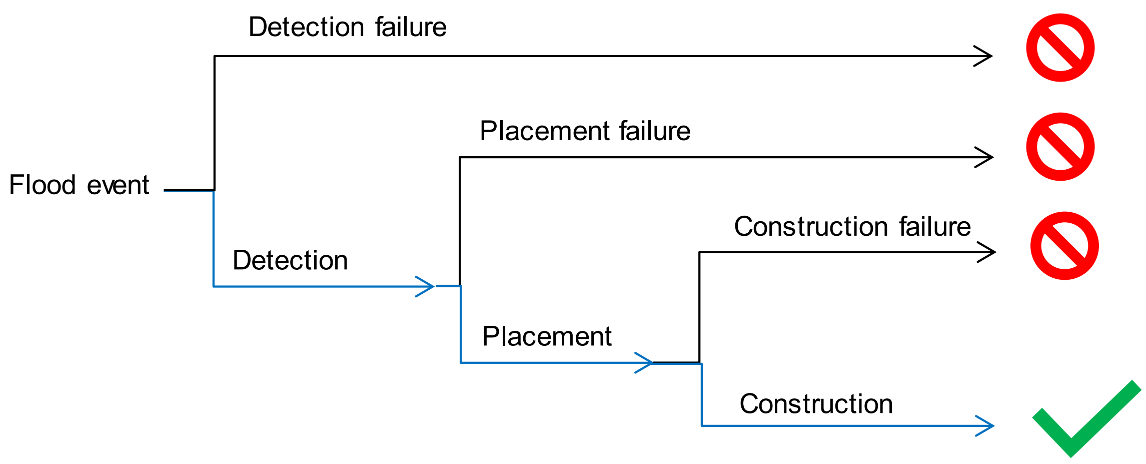

Figure 22: Event tree for implementation of emergency measures

Failure to perform the required tasks correctly

Human errors play an important role in the 'Detection' and 'Placement' sub-events. According to Corn and Inkabi (2013), human reliability practitioners have to rely on expert judgement in combination with limited numerical data to determine the probability of human errors, due to lack of a successful database of human error probabilities. Most human reliability studies, such as (Bea 1998; Rasmussen 1983), typically seek only orders of magnitude of error probabilities rather than exact descriptions. In these studies, the qualitative analysis of the system is the most important aspect (Rasmussen 1982).

To estimate typical failure rates of the 'Detection' and 'Placement' phase for emergency measures, the method of Rasmussen is applied here. Rasmussen uses a generic psychological classification of human behaviour and corresponding error rates which can be applied to specific task performances (Rasmussen 1982). The relation between common error rates and three performance levels is shown in the following figure, and explained below. The figure also shows a classification of main stakeholders involved in the implementation of emergency measures for river flood defences, based on a case study treated in Section 3.3.

This model distinguishes between three levels of behaviour: Knowledge-based, Rule-based and Skill-based behaviour (Rasmussen 1983):

- **Knowledge-based performance** is the most cognitively demanding level; at this level there are no pre-planned actions which can be called upon because of the novelty of the situation. The assessor is required to analyse the unfamiliar situation, develop alternative (conceptual) plans and choose the plan which is considered to be the best alternative (Rasmussen 1983). Error rates vary between $5 \cdot 10^{-1}$ and $1 \cdot 10^{-2}$ per task.
- **Rule-based performance** is the next cognitive level; this level involves responding to a familiar problem according to standardized rules. The rule to be applied is selected from previous successful experiences (Rasmussen 1983). The error rates vary between $5 \cdot 10^{-2}$ and $5 \cdot 10^{-4}$ per task.
- **Skill-based performance** is the least cognitively demanding level; at this level the calling conditions occur so often that knowledge retrieval and action are virtually automatic. Normally, skill based performance occurs without conscious attention or control (Rasmussen 1983).The error rates vary between $5 \cdot 10^{-3}$ and $5 \cdot 10^{-5}$ per task.

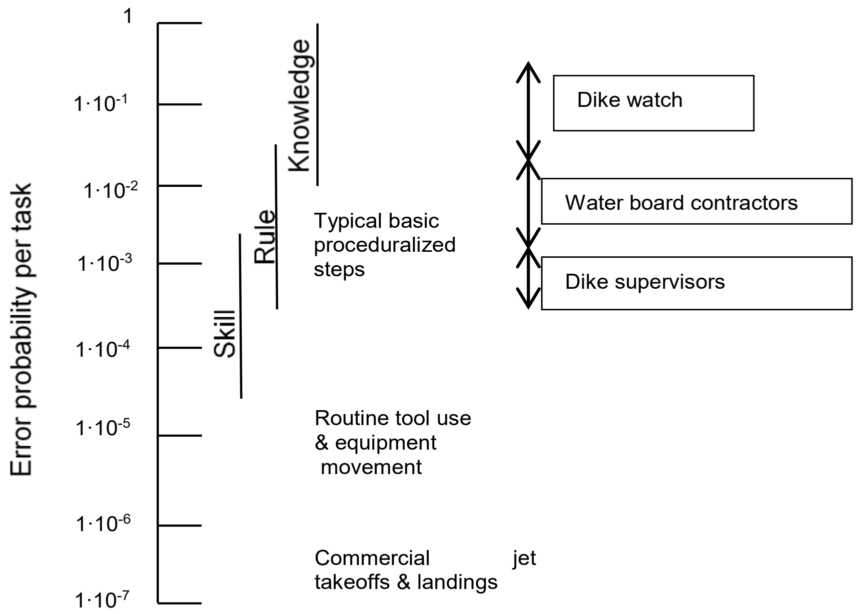

Figure 23: Human error probabilities and performance levels by Watson and Collins (Bea 2010)

The main stakeholders involved in the application of emergency measures are the dike watch, for inspection of the flood defence during river floods; contractors, for placement of the emergency measures; and dike supervisors, who are employees responsible for day to day maintenance of flood defences. These are different stakeholders who work together, but each have a distinct role in implementing emergency measures. Each stakeholder can be classified in one of the performance levels, based on examination of tasks, interviews, observations and expert judgement. For example, the dike watch consists of volunteers who receive instruction on the inspection of dikes every two years. We concluded that their performance level is knowledge-based, because of their low experience with inspections making this an unfamiliar situation. In comparison, the performance level of a dike supervisor is categorised as skill-based, because they are confronted with dike maintenance on a daily basis (see Figure 23 and (Lendering et al. 2013) for further discussion).

Failure to complete the required tasks within the available time

The second failure mode of the 'Detection' and 'Placement' sub-events is failure to complete the detection and placement of the emergency measure due to insufficient time. Insufficient time occurs when the available time, defined as the time until flood defence failure, is less than the required time, defined as the time required for detection and placement of the emergency measures. Factors which determine the required time are the capacity of the organization (personnel, equipment and material), the travel distances, the weather conditions and the speed of detection and/or placement. The time line for detection and placement of emergency measures is illustrated in Figure 24.

For overflow, failure occurs when the water level is higher than the retaining height of the flood defence, which will occur during the peak of the expected water level. This peak can be predicted hours (e.g. storm surge / rain) to days in advance (e.g. river floods), depending on the hydrologic system under consideration. The accuracy of the predicted water levels increases as the peak of the river flood approaches. Contrary to overflow failure, piping failure can occur at lower water levels, before the peak of the river flood arrives, but can also be delayed due to the build up of water pressures in the aquifer. These time dependencies depend on the strength of the considered flood defence.

The available time is compared to the required time. With this comparison we determine the probability of having insufficient time to complete detection and placement. The required time consists of the summation of the detection time (T_d) and the placement time (T_p) in hours, see Eq. 3.3. Failure occurs when the required time is larger than available time, see Eq. 3.4. The probability of insufficient time is determined probabilistically, with Monte Carlo simulations of Eq. 3.4. To account for the uncertainties in the flood prediction and required time, we assume normal distributions for the available time (T_a) and the required time (T_r), see Figure 25.

A similar method is used in (Frieser 2004) to determine the probability of complete evacuation of people within the available time, in this example normal distributions are also used for the available and required time. Moreover, according to central limit theorem, the sum of a large number of independent variables will be approximately normally distributed.

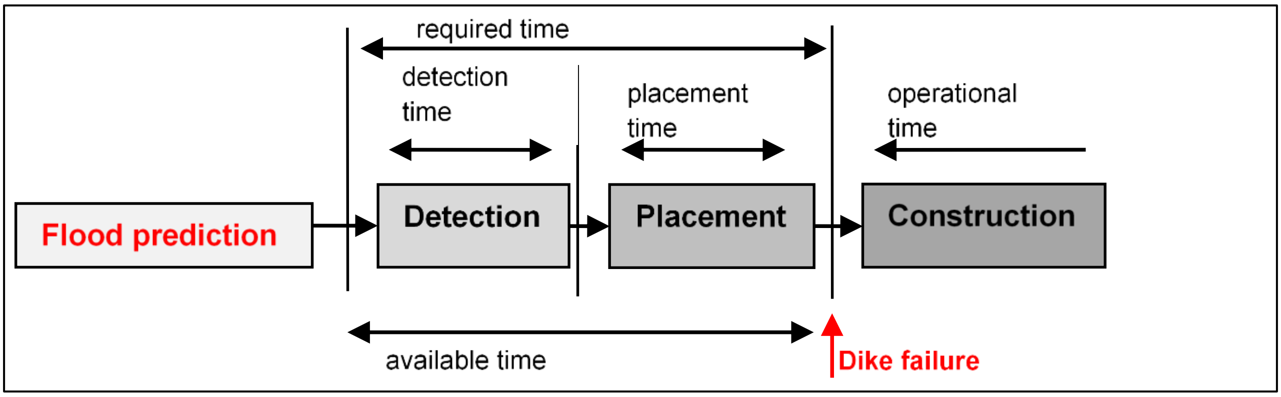

Figure 24: Time line emergency measures

$$T_r = T_d + T_p \quad (3.3)$$

$$Z = T_a - T_d - T_p \quad (3.4)$$

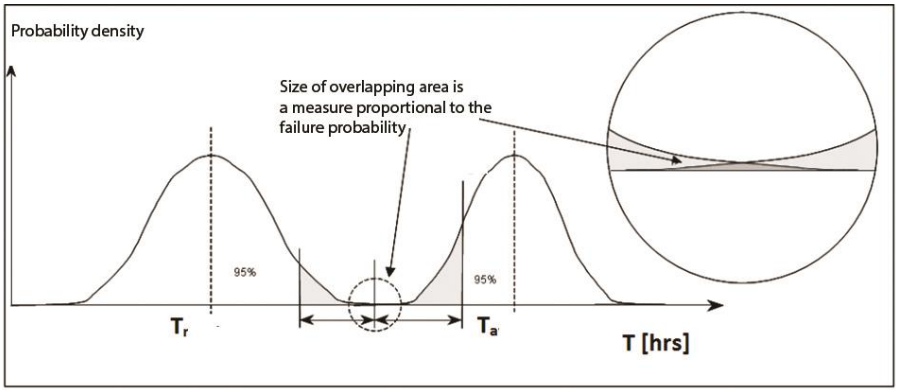

Figure 25: Probability density of the required time (T_r) versus the available time (T_a)

Structural failure of the emergency measures ('Construction')

Structural failure of the emergency measures is the failure mode of the 'Construction' sub event. This subsection explains how to determine the probability of structural failure of emergency measures used to repair weak spots for overflow and piping. First a description is given of the types of emergency measures which can be used for these failure modes. When a dike is threatened by overflow, two types of emergency measures can be used to prevent breaching: either the retaining height is increased locally with a temporary water retaining structure to prevent overflow or the inner slope of the dike is protected against erosion while allowing overflow. In this chapter, we consider the temporary water retaining structures that raise the elevation of the dike, because these aim to prevent overflow.

The first signs of piping failure are the development of boils and/or sand boils on the inner side of the dike, see (Schweckendiek et al. 2014). To prevent sand boils from growing, containments are built around the boils with temporary water retaining structures, typically consisting of sand bags. These containments fill with seepage water providing counter pressure and thus reducing the flow velocity and further erosion of sand particles. When there are many sand boils along a certain dike section, larger scale measures are taken. Examples are water or soil berms; these reduce the water head and provide extra stability at the toe of the dike. Similar measures are used to increase the stability of the inner slope of the dike.

Summarizing, for the considered failure modes of dikes, which are overflow and piping, temporary water retaining structures are used to increase the retaining height locally, or to construct sand boil containments. Authorities still largely rely on the 'traditional' sand bag for these purposes, even though new products are available such as boxes or tubes filled with water. The cross section of a structure of sand bags is built in a pyramid shape; each subsequent deeper layer consists of one sand bag more than the latter. Figure 26 shows a schematic overview of loads on a schematized sandbag structure.

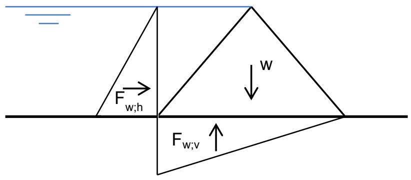

Figure 26: Typical loads on a schematized temporary water-retaining structure consisting of layers of sand bags.

These structures can be seen as small gravity structures, which obtain stability through their own weight (W). The loads consist of the horizontal water pressure ($F_{w;h}$) and upward water pressure ($F_{w;v}$). The stability is largely influenced by the weight and the development of upward water pressure under the structure, which depends on the subsoil, loading time and connection between the structure and the subsoil. The failure mechanisms of these structures are similar to those of flood defences, overflow, instability due to sliding or rotation and seepage (Figure 27), and occur independently of failure of the flood defence.

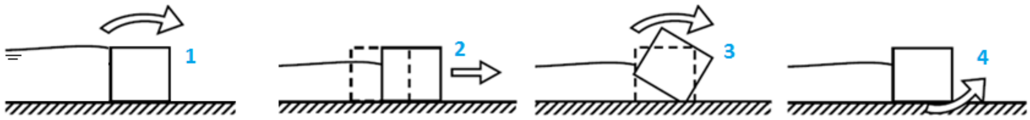

Figure 27: Typical structural failure mechanisms of temporary water retaining structures: Overflow (1), Sliding (2), Rotation (3) and Seepage (4) (Boon 2007)

Assuming that these structures are constructed on impermeable subsoil on the outer layer of flood defences, instability due to sliding will be the governing failure mode (Boon 2007). We neglect the upward water pressure due to the temporary nature of the load and the construction on impermeable clay layers. Sliding occurs when the horizontal force on the structure exceeds the friction force between the structure and the subsoil due to self-weight, see Eq. 3.5.

$$Fs_{sliding} = \frac{f \cdot (\mathrm{W} - \mathrm{F}_{\mathrm{w;v}})}{F_{w;h}} > 1 \qquad (3.5)$$

$Fs_{sliding}$ represents the safety factor for sliding, stability is obtained when $Fs_{sliding} > 1$ (-). To gain more insight into the sliding failure mode of sandbags, experiments were conducted (see (Lendering et al. 2013)). Sliding can occur between each subsequent layer of sand bags or between the bottom layer of sand bags and the subsoil. The interface where sliding failure occurs depends on the friction force between two layers of sandbags, or between sand bags and subsoil, see (Krahn et al. 2007). Probabilities of failure are estimated using Monte Carlo simulation of Eq. 3.5. Input data based on typical emergency measures and dike data is given in Table 11.

Variable	Parameter	Distribution	Equation	Value	Unit
Ø	Friction angle of subsoil (clay)	Deterministic	-	$\mu = 24$ $\sigma = 2$	°
H_r	Retaining height structure	Normal	-	$\mu = 0.5$ $\sigma = 0.05$	m
B	Width of structure	Normal	-	$\mu = 1$ $\sigma = 0.1$	m
y_s	Volumetric weight sand bags	Deterministic		26.5	kN/m^2
y_W	Volumetric weight water	Deterministic		10	kN/m^2
$F_{w;v}$	Upward water pressure	Deterministic		0	kN/m
F	Friction coefficient between sand bags and subsoil	Deterministic	tan(Ø)		-
V	Volume of pyramid structure	-	$0.5 \cdot B \cdot Hr$		m^2
W	Weight of structure	-	$V \cdot y_s$		kN/m
$F_{w;h}$	Horizontal force	-	$0{,}5 \cdot y_w \cdot H_r^2$		kN/m

Table 11: Input data for structural failure probabilistic calculations

3.2.3 Failure of the flood defence given successful implementation of emergency measures

The failure probability of the flood defence system without emergency measures is determined with probabilistic methods. For this purpose, the flood defence system is divided in sections with similar strength properties. For each section the failure probability is determined, after which the failure probability of the flood defence system is calculated by combining the individual sections, taking dependencies into account. This method is applied to determine the failure probability of all flood defence systems in the Netherlands in 'project VNK', more details can be found in (Vrijling 2001), (Jongejan & Maaskant 2013) and (VNK 2005) but also internationally in (IPET 2009) and (Vorogushyn et al. 2009).

The same method is also used to determine the failure probability of the flood defence system given successful implementation of emergency measures, by simulating the increased strength properties of the considered section with the implemented emergency measure. For example, placing temporary water retaining structures on top of a river dike is taken in to account by modelling an increase of the retaining height of the considered section. This will result in a lower probability of failure than without the temporary structure, due to the increase of strength for overflow.

The extent of the reduction of the failure probability depends on the type of measure and the failure mode of the flood defence. The conceptual fragility curves in Figure 28 show the failure probability [P(f|h)] of the flood defence conditional on the water level, for the governing failure modes overflow and piping. It illustrates the potential increase in reliability for both failure modes after successful implementation of an emergency measure. As can be seen, temporary measures for overflow mainly reduce the failure probability of the section for water levels close to the crest, while piping measures can potentially reduce the failure probability at lower water levels. This is mainly due to the original shape of the fragility curve.

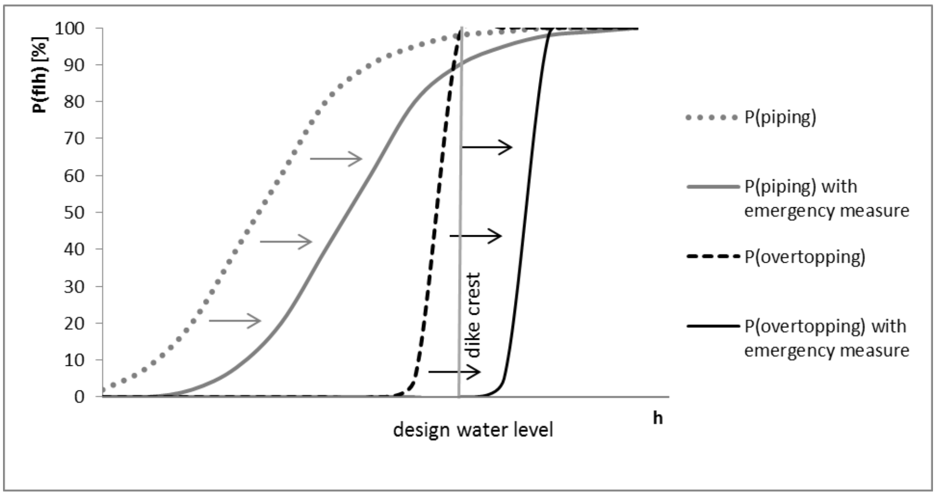

Figure 28: Example of a typical fragility curve for river dikes

Note that emergency measures meant to increase the reliability of one failure mode can have a negative effect on the reliability of another failure mode. For example, a temporary water retaining structure (e.g. sand bags) on top of a flood defence will increase the retaining height, and therefore the head over the flood defence; as a result, the water pressure under the blanket layer is increased. This may lead to more seepage and an increased risk of piping. Therefore, an assessment of the effect of the emergency measure on all failure mode is required before emergency measures are placed.

3.2.4 Combining the failure probability of the flood defence and the emergency measure

The sub events which determine the reliability of the emergency measure (Figure 22) are integrated with the general event tree (Figure 21) to form an integrated event tree (Figure 29). The integrated event tree is used to determine the failure probability of a flood defence system with implementation of emergency measures. Failure due to insufficient time during ‘Detection’ and ‘Placement’ is considered a separate failure mode, which is modelled in the integrated event tree with a separate sub event. The variables included are explained below:

- Probability of human error during ‘Detection’ [P_d];
- Probability of human error during ‘Placement’ [P_p];
- Probability of insufficient time for ‘Detection’ and ‘Placement’ [P_t];
- Probability of structural failure during ‘Construction’[P_c].

Based on these variables the probability of failure of emergency measures can be calculated as follows, assuming independency between each failure mode in the series system of sub events:

$$P(E) = 1-[(1-P_d) \cdot (1-P_p) \cdot (1-P_t) \cdot (1-P_c)]; \qquad (3.6)$$

Consequently, one has to take into account:

- Probability of failure of the dike, given failure of emergency measures [P(D|E)];
- Probability of failure of the dike, given successful implementation of emergency measures [P(D|Ec)].

The only combination of events in which the emergency measure is successfully implemented reducing the failure probability of the flood defence is the bottom branch, shown in Figure 29 in blue. In all other branches, the failure probability of the dike without emergency measures has to be taken in to account, as the emergency measure is not successfully implemented. Consequently, the difference between P(D|E) and P(D|E^c) is a stronger (or higher) dike due to the emergency measure.

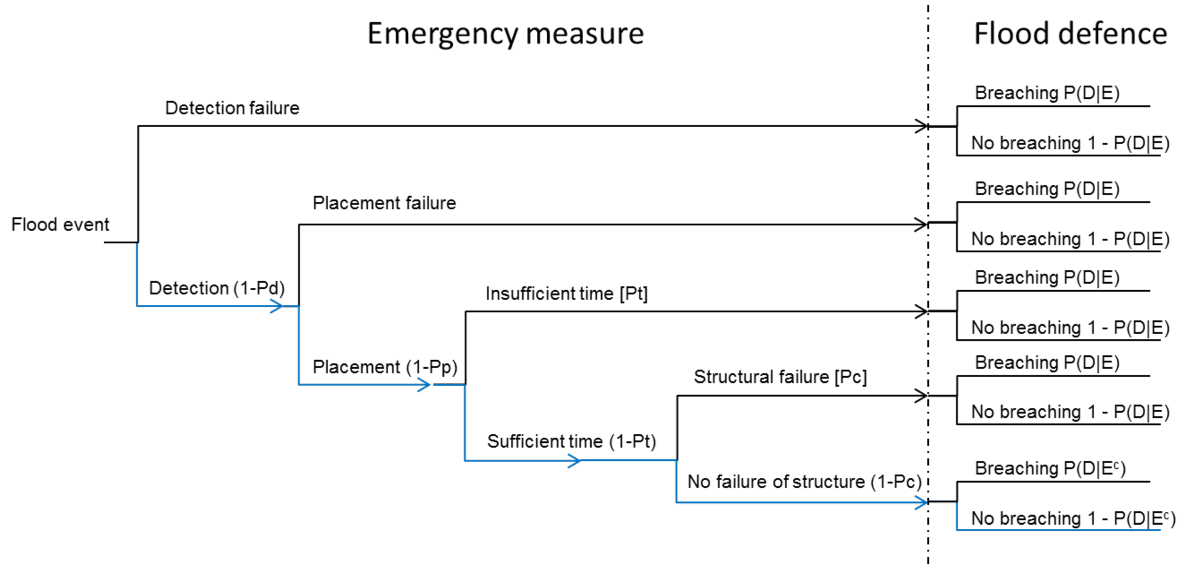

Figure 29: Integrated event tree for assessing the reliability of a dike section with emergency measures.

This method can also be applied to a flood defence system consisting of multiple sections. To determine the failure probability of the flood defence system, all sections are modelled as a series system. The failure probability of the flood defence system can be determined when insight is obtained into the dependencies between different sections. Flood defence sections subject to overflow are modelled dependently, because the loads and the retaining height are similar along the flood defence. Thus, it can be assumed that if one section overflows, it is most probable that the next section will also overflow. However, sections subject to piping are modelled independent, because of the large uncertainties and high variability in the subsoil. As a result, piping will have a large 'length effect' (VNK 2005): longer flood defence systems will have higher failure probabilities for piping than shorter flood defence systems.

Emergency measures have a similar 'length effect' in the detection phase, i.e. their failure probability depends on the amount (or length) of emergency measures required. The level of dependence depends on the failure mechanism considered. It is assumed that a dike watch who detects an overflowing section will also find other sections subject to overflow, because differences in height are clearly visible. Hence, allowing for dependence between subsequent dike sections. However, weak spots for piping (e.g., sand boils) are much more difficult to detect; the detection of one sand boil in one dike section is no guarantee for finding the next. Hence, for piping, length effect is taken in to account by assuming that the probabilities assessed with Figure 23 represent one dike section, requiring a summation taking independence between subsequent sections in to account.

3.3 Case study 'Salland' in the Netherlands

3.3.1 Introduction

In the Netherlands, flood prone areas are divided in dike rings; these are rings of flood defences which protect the low lying areas (VNK 2005). The regional water authorities, also called 'water boards', are responsible for the maintenance of these dike rings. A case study was undertaken for a Dutch water board, near the city of Zwolle in the Netherlands. The studied dike ring, named 'Salland', can be seen in Figure 30. The dike ring protects the area from flooding from the river Ijssel, which is part of the river Rhine and flows in to the Ijssel lake in the northern part of the Netherlands. The dikes consist of a sandy core covered by an impermeable clay layer. The current failure probability, determined by project VNK, is estimated by means of probabilistic analysis where different failure modes are considered and local data and information is used as input. This resulted in a failure probability larger than 1/100 per year (Dijk & Plicht 2013) with contributions from overflow, piping, inner slope instability and outer slope erosion. This chapter considers piping and overflow, which have failure probabilities of 1/60 per year and 1/600 per year respectively. The failure probability of piping is relatively high because of a narrow dike profile and permeable subsoil. The water board acknowledges that piping failure is a relevant issue along these dikes as numerous sand boils often form during high river discharges, without having breached.

The water board receives predictions of river floods several days in advance. Depending on the expected river discharge inspections of the dikes along the river are made by the water board. When extreme river discharges are expected permanent inspection is required, for which the water board does not have sufficient capacity. In this case, the dike watch is asked to perform the inspections, which consists of volunteers who periodically (once every two years) receive a course on dike inspection. During their inspection they are required to report possible weak spots to the dike supervisors of the water board. After receiving a report of a weak spot, they decide upon the implementation of emergency measures, which are will then be implemented by a contractor. Through interviews and observations performed by the authors, estimates were made of the performance levels of these stakeholders for different failure mode. These estimates assume the results are shown in Figure 23 and Table 12.

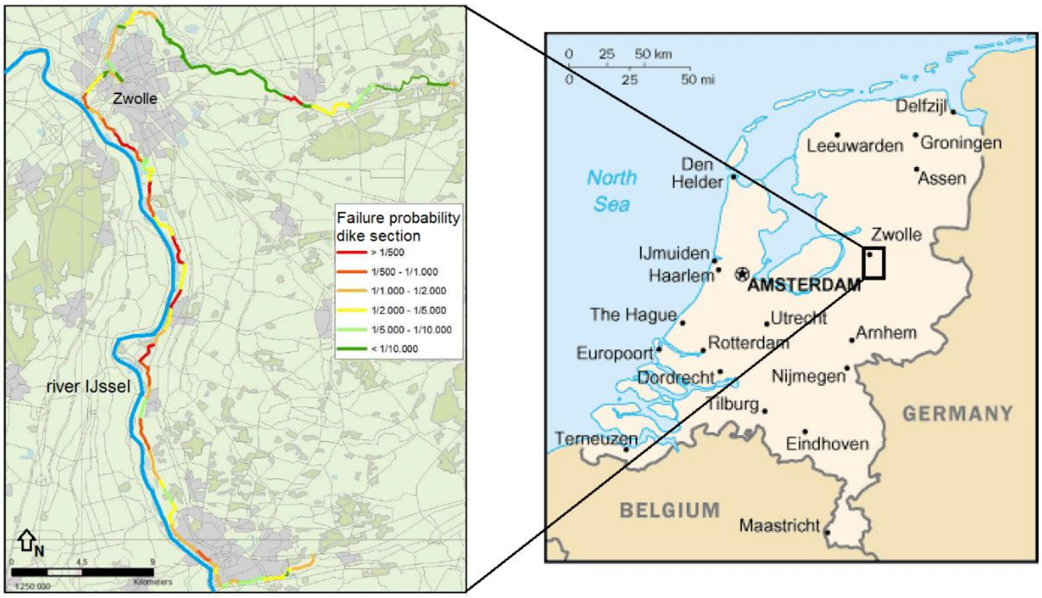

Figure 30: Case study area with failure probabilities of flood defences (left) (Dijk & Plicht 2013)

Variable	Parameter	Overflow value (event-1)	Piping value (event-1)	Source / Explanation
Detection	P_d	1/10	1/3	Human error probability. Inspection is done by the dike watch.
Placement	P_p	1/20	1/20	Human error probability. Implementation of emergency measures is done by the contractors, who have more experience than the dike watch.
Insufficient time	P_t	1/100	1/200	Analysis of available and required time, see below.
Construction (structural failure)	P_c	10^{-4}	10^{-4}	Reliability analysis of temporary structure.

Table 12: Failure probability of emergency measures in case study

The probability of failure of detection for piping is higher than for overflow, because overflow detection is modelled dependent whereas piping detection is modelled independent. A maximum of four independent sections is assumed for each operator in the Detection phase, resulting in a maximum length effect of about 3,3 [$P_{f;d} = 1-(1-P_f)^4$].

The probability of failure of complete placement within the available time was determined based on the method presented in Section 3.2. Data on the amount of time required for emergency measures was made available by the water board. Dutch river systems are more vulnerable to piping failure than to overflow failure (Kanning

2012). In the early stages after prediction of a river flood more attention is paid to finding weak spots for piping than for overflow. At a later stage, when the predictions of expected water levels are more accurate, and overflow threatens to occur, attention will be paid to measures which prevent overflowing. The available time was assumed to be 4 days for piping and 2 days for overflow. For more information regarding these calculations reference is made to the technical report (Lendering et al. 2013).

Together with project VNK (Dijk & Plicht 2013), sensitivity analyses were made to determine the failure probability of the dike ring with successful implementation of emergency measures for both overflow and piping. These were made for the ten weakest dike sections of the flood defence ring. For overflow we determined the effect of increasing the retaining height of local ‘dents’ in the dike. The maximum length of the ‘dent’ was assumed to be 250 meters. When the retaining height of these dents was increased, the failure probability reduced from 1/600 to 1/3,600 per year. For piping, the effect of reducing the water head was determined, since the water head is the main load for the piping failure mode. With a reduction of 0.5 meter water head over the flood defence, the failure probability reduced from 1/60 to 1/150 per year. Note that this is the potential reduction of the failure probability with successful implementation of emergency measures, which does not include the failure probability of the emergency measure.

3.3.2 Case study results

Using the input of the case study we were able to calculate the failure probability of both overflow and piping emergency measures. Consequently, we found that the failure probability of emergency measures for overflow was 1/9 per event. For piping emergency measures, we calculated a failure probability of 1/3 per event. The distribution of factors which determined the failure probability of both piping and overflow emergency measures (P(E)) are shown in the pie charts in Figure 31.

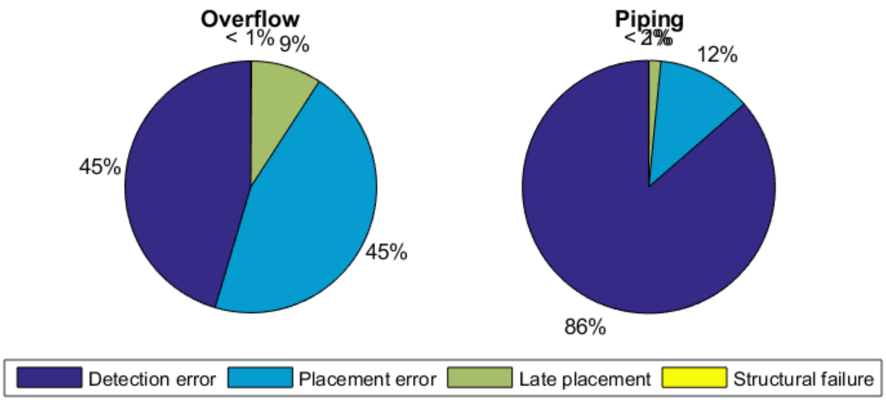

Figure 31: Contributions of factors that determine the failure probability of the emergency measures for overflow and piping.

The charts show that human errors during the ‘Detection’ and ‘Placement’ phases account for over 90% of the failure probability of emergency measures. Especially errors during ‘Detection’ proved to be dominant for piping emergency measures, because weak spots for piping are difficult to find. In comparison, the failure probability of overflow is lower, as detection of weak spots is more straightforward for overflow. The charts also show that failure due to insufficient time only partly contributes to the failure probability of piping emergency measures, but is more dominant in the case of overflow emergency measures. This can be explained because overflow weak spots generally require a larger length (several hundred meters), whereas piping emergency measures are very local (only a number of sand boil containments).

The estimated failure probabilities of structural failure are in the order of 10^{-4} per event, which is smaller than typical failure probabilities of human error and/or failure due to insufficient time (order of 10^{-1} per event). Structural failure is therefore negligible when including the probability of the sub events ‘Detection’ and ‘Placement’. Here we only considered a structure of sand bags as a temporary water retaining structure; if another type of emergency measures is implemented the same method can be used to determine its failure probability.

By means of event tree analysis (see Figure 29), the failure probability of the whole system was determined, the results are shown in Table 13. For overflow, the resulting failure probability of the system was reduced to 1/2,300 per year, which is a reduction of a factor 4. For piping, the failure probability of the system was reduced to 1/100 per year, which is a reduction of almost a factor 2.

Variable	**Parameter**	**Overflow**	**Piping**
Failure probability of emergency measure	P(E)	1/9 event^{-1}	1/3 event^{-1}
Failure probability of dike ring without emergency measure	P(D|E)	1/600 yr^{-1}	1/60 yr^{-1}
Failure probability of dike ring with emergency measure	P(D|E^c)	1/3,600 yr^{-1}	1/150 yr^{-1}
Failure probability of the system	P_f	1/2,300 yr^{-1}	1/100 yr^{-1}

Table 13: Case study results

3.3.3 Sensitivity analyses

In this paragraph we discuss several sensitivity analyses to discuss the assumptions made in the method and some of the inputs for the case. Increasing the reliability of the organization and people involved will reduce the failure probability of emergency measures. The contribution of human errors, in the detection and/or placement phase (P_d and/or P_p), to the total failure probability of emergency measures is shown in Figure 32. The probability of these human errors is plotted on the horizontal axis against the total failure probability of emergency measures (P(E)) on the vertical axis. The figure shows that reductions of the human error probability can be very effective, up to a failure probability of 10^{-2} per event. This level corresponds with the performance level of dike supervisors. Higher reductions will have a small effect on the total failure probability of the emergency measures, because the probability to complete detection and placement within the available becomes dominant.

The probability human errors can be reduced through training programs for specific levels of flood responders (dike watch and dike supervisors), which increase improving the responsive skills of all stakeholders involved in flood fighting. It is of vital importance that stakeholders develop routine skills, on a skill based level, to effectively perform their tasks during the actual river flood and prevent human errors. Providing help in other countries during large flood events can further contribute to developing these skills, as well as flood simulations and training centres. In addition, innovative detection methods, such as remote sensing and the use of drones, could also increase the reliability of the 'Detection' phase in the future. This requires further investigation.

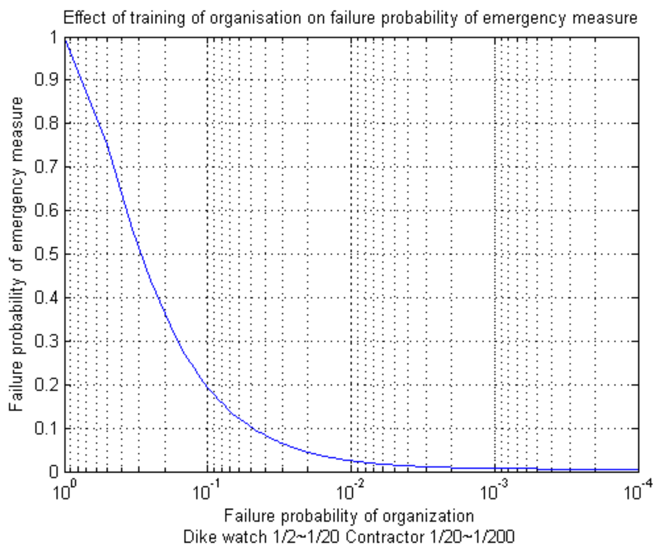

Figure 32: Influence of human reliability on total failure probability of the emergency measures

3.4 Comparison of flood risk reduction strategies

3.4.1 Approach

As a final step in this chapter we will compare emergency measures with traditional flood defence reinforcements, to determine which strategy is more cost-effective. In the Netherlands, about one thirds of the flood defences currently do not meet the flood safety standards. Large investments are required to reinforce these flood defences. The traditional method to reinforce flood defences is raising the retaining height or constructing piping berms. This chapter has shown that emergency measure can also help to reduce the failure probability of a flood defence. However, the previous sections have not considered the costs of the emergency measures. To justify implementation of emergency measures, or any other risk reduction strategy, cost benefit analyses are required. Cost benefit analyses have long been used in the Netherlands to inform policy debates about the current and optimal safety standards of flood defences (Jongejan et al. 2012). In this section, the costs and benefits of both strategies are compared to the current situation, see Figure 33.

	Strategy	Reduction of risk	Unit Cost [€]
	Do Nothing	None	0
Flood defence	Emergency measure	Factor 1.5~3	~ 100,000 km^{-1} yr^{-1}
	Reinforcing flood defences	Factor 10	~ 5 mln km^{-1}

Figure 33: Risk reduction strategies for a system which does not meet the safety standards

To compare the different strategies the net present value of the total cost (TC [€]) is determined, which is expressed as the summation of the initial investment (I_0 [€]), the operational cost (C_{annual} [€]) and the flood risk (R [€]). Flood risk is defined as the annual expected damages, which is found by multiplying the probability of flooding (P_f [yr^{-1}]) with the damage of flooding (D [€]). The benefits are expressed as the avoided damages, in other words the reduction of the annual flood risk. To determine which option is economically the most attractive, the total cost of all options is calculated for an infinite time horizon. The net present value of the annual operational cost and flood risk is found by dividing by the discount rate (*r [-]*), which is estimated at 5.5% according to (Deltares 2011).

$$TC = I_0 + C_{annual} + R \ [€] \qquad (3.7)$$

$$TC = I_0 + \frac{C_{annual}}{r} + \frac{R}{r} \qquad \text{with } R = P_f \cdot D \qquad (3.8)$$

3.4.2 Case study results

The total cost of reinforcements and emergency measures are compared to the current 'Do Nothing' scenario for the Salland case study. As shown in the case study, the emergency measures can increase reduce the failure probability of the flood defences by a factor 1.5 to 3. In comparison, it is assumed reinforcements will reduce the failure probability to the required safety standard, which is a reduction of about a factor 20. However, in practice reinforcements will reduce the failure probability further than the required safety standard to account for sea level rise, settlements and/or subsidence. In the case study the safety standard is assumed to be a maximum failure probability of 1/1,250 per year. Table 14 shows the variables used in the cost benefit analysis, which are described in more detail below.

Variable	Parameter	Value
Length of the dike ring	L	10 km
Discount rate	r	5.5%
Initial failure probability dike ring	$P_{f;initial}$	1/60 yr-1
Damage potential during flooding	D	1010 €
Failure probability with reinforcement	$P_{f;reinforced}$	1/1,250 yr-1
Initial investment of reinforcement	$I_{reinforcement}$	5 ·106 €/km
Operational cost reinforcement	$C_{annual;reinforced}$	0
Failure probability with emergency measures	$P_{f;emergency\ measure}$	1/180 yr-1
Operational cost emergency measures	$C_{annual;em;training}$	5 ·105 €
Flood event cost of emergency measures	$C_{annual;em;flood\ event}$	1 ·105 €/km

Table 14: Case study parameters

The investment of reinforcement is 5 million euro per kilometre; no operational costs are assumed as the flood defence will comply with the safety standard after reinforcing. For emergency measures, no initial investment is required. The operational cost is divided in cost for the training program and cost for implementing emergency measures during a flood event. The cost for training is about 50,000 euro per year. The cost for implementing emergency measures during a flood event is 100,000 euro per kilometre, or 1 million euro for the whole dike ring. The annual probability of having to implement these emergency measures is assumed to be 1/10. The annual cost of implementation is found by multiplying the probability of having to implement the measures with the cost of implementation, which results in an annual expected cost of 100,000 euro. The total cost for all options are shown in Table 15.

Measure	**Flood probability [yr^{-1}]**	**Initial investment [€]**	**Operational cost [€/yr]**	**Risk [€/yr]**	**Cost [€]**	**Risk [€]**	**Total cost [€]**
Do nothing	1/60	0	0	$1.7 \cdot 10^8$	0	$2.8 \cdot 10^9$	$2.8 \cdot 10^9$
Emergency measures	1/180	0	$5.1 \cdot 10^5$	$0.6 \cdot 10^8$	$0.9 \cdot 10^7$	$0.9 \cdot 10^9$	$1.0 \cdot 10^9$
Flood defence reinforcement	1/1,200	$5.0 \cdot 10^7$	0	$0.1 \cdot 10^8$	$5.0 \cdot 10^7$	$0.1 \cdot 10^9$	$0.2 \cdot 10^9$

Table 15: Example cost-effectiveness emergency measures versus reinforcement

Both emergency measures *and* flood defence reinforcement reduce the total cost when compared to the current 'Do Nothing' situation and thus have a benefit / cost ratio larger than 1. Traditional reinforcements are most effective but requires a large initial investment. Emergency measures can be an economically attractive compared to the current 'Do Nothing' situation. During a period that reinforcements are delayed due to budget constraints, or in preparation since these take several years to complete, emergency measures can serve as an interim solution to reduce flood risk.

3.4.3 Sensitivity analysis

The cost-effectiveness of these strategies depends highly on the initial failure probability of the dike ring, which is shown in Figure 34. The figure shows the total cost of the options related to the initial failure probability of the dike ring. In the example discussed in the last section, traditional reinforcement and emergency measures have lower total cost than the 'Do Nothing' option due to an initial failure probability of 1/100 per year. Similar conclusions are drawn for initial failure probabilities between 1/100 to 1/1,000 per year. Below 1/1,000 per year, emergency measures have lower total cost than traditional reinforcement. However, the total cost of emergency measures (and traditional reinforcements) is of the same order of magnitude than the cost of doing nothing. Meaning that no substantial risk reduction is achieved with either of these options. In conclusion, for initial failure probabilities below 1/1,000, no significant reduction in total costs is achieved with the considered options. This is further illustrated with Table 16.

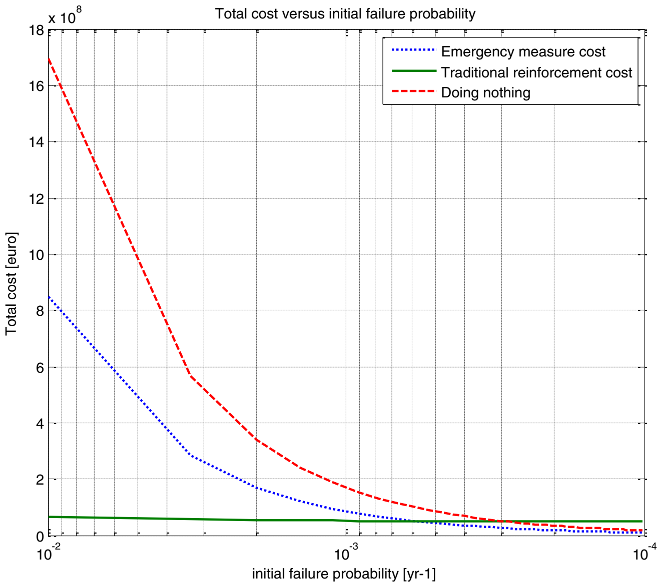

Figure 34: Total cost flood risk reduction strategies versus the initial failure probability of the flood defence system

The following table illustrates the total costs for each option, for different initial failure probabilities.

	Initial failure probability [yr^{-1}]			
	1/10	**1/100**	**1/1,000**	**1/10,000**
Total cost Doing nothing [€]	$1.7 \cdot 10^{10}$	$1.7 \cdot 10^{9}$	$1.7 \cdot 10^{8}$	$0.1 \cdot 10^{8}$
Total cost Emergency measures [€]	$0.9 \cdot 10^{10}$	$0.9 \cdot 10^{9}$	$0.9 \cdot 10^{8}$	$0.1 \cdot 10^{8}$
Total cost Traditional reinforcement [€]	$0.2 \cdot 10^{10}$	$0.7 \cdot 10^{8}$	$0.5 \cdot 10^{8}$	$0.5 \cdot 10^{8}$

Table 16: Total cost for doing nothing, emergency measures and traditional reinforcements, given different initial failure probabilities

3.5 Conclusions and recommendations

3.5.1 Conclusions

A method has been developed which can be used to determine the reliability of a permanent flood defence system including emergency measures. In addition, we showed how the results of this method can be used to determine the most cost-effective flood risk reduction strategy; emergency measures or traditional reinforcements. The reliability of emergency measures depends on the probability of human errors, the probability of insufficient time for implementation of emergency measures during ‘Detection’ and ‘Placement’ and the probability of structural failure of the emergency measure during ‘Construction’. The reliability of a flood defence system with emergency measures depends on the considered failure mode of the flood defence.

Overall, we conclude that the reliability of flood defences systems with high initial failure probabilities can be increased cost-effectively with emergency measures. However, traditional flood defence reinforcements remain more cost-effective for these flood defence systems. For high risk locations, we suggest including emergency measures as an interim solution to temporarily reduce flood risk, while preparations or construction of flood defence reinforcements are undergoing.

3.5.2 Case study

The method was applied to a case study at a water board in the Netherlands, in which the failure probability of a dike ring along a river system was determined with and without emergency measures. The calculated failure probability of overflow emergency measures was 1/9 per event. After including the emergency measures in the flood defence system, the failure probability reduced by a factor 4. For piping emergency measures, a failure probability of 1/3 per event was calculated, which reduced the failure probability of the flood defence system by a factor 2. The difference between overflow and piping is explained by the difficulty of the 'Detection' phase for piping, which resulted in higher failure probabilities for this phase. The failure probability of the emergency measures was determined largely (>90%) by human errors during 'Detection' and 'Placement'. Failure due to insufficient time only partly contributes to the failure probability of the emergency measures, whereas structural failure probability of the emergency measure is negligible.

The extent to which the failure probability of the flood defence system is reduced strongly depends on the probability of human errors, the available time and the considered failure mode of the flood defence. Given the series system of the emergency measures, large reductions of the failure probability of emergency measures are difficult due to significant contribution of human error probabilities.

The case study demonstrated that both emergency measures and flood defence reinforcements can be cost-effective for high initial failure probabilities, with reinforcements being more cost-effective. For the long term, reinforcements are preferred from an economical point of view. During a period that reinforcements are delayed (e.g., due to budget constraints, or in preparation, since these take several years to complete), emergency measures can serve as an interim solution to reduce flood risk. Note that the preferred strategy strongly depends on the initial failure probability of the dike ring. For small initial failure probabilities, the total costs of both options was in the same order of magnitude as doing nothing, resulting in the conclusion that no significant reduction of risk is obtained with either option.

3.5.3 Recommendations

The results of the case study strongly depend on local parameters and input. The assignment of error rates to the tasks performed by each stakeholder was based on expert judgement and literature, which were not proven to be false during a large-scale exercise at the water board of the case study. However, there is limited experience with human reliability analyses in flood defence systems. Further investigation of human tasks in these and other systems is recommended, which can provide more insight into the validity of the error rates. In addition, we recommend investigating how human and organizational failure probabilities can be reduced.

We focussed on emergency measures in river systems with prediction times of 2 to 4 days. In a coastal flood protection system, floods can occur due to storm surges or wind set up, which have much shorter prediction times than the considered river system. Overflow and overtopping failure of the flood defence are most likely to be dominant in these systems, rather than piping. As a result, the required versus available time is much more dominant and will most likely result in higher failure probabilities. This requires further investigation.

The method included a binary approach to combine the reliability of emergency measures with the reliability of the flood defence system for specific failure modes. However, emergency measures for one failure mode of a flood defence system may have a negative effect on another failure mode. It is recommended to assess the effect of emergency measures on all failure modes of the considered flood defence before these are implemented to assure that the overall failure probability is reduced. A more advanced probabilistic method, wherein all emergency measures and failure modes of the flood defence system are considered, will help to show the effect of singular emergency measures on all failure modes of the flood defence system.

Event and fault trees grow rapidly with increasing number of variables / factors to take in to account. While these methods worked well for the (limited) variables taken in to account in this approach, it is recommended to consider other tools for reliability analyses when the factors that determine human error probabilities and the interdependencies with other failure mechanisms are taken in to account. Examples of such tools are Bayesian Networks and N2 diagrams. The method is based on the Dutch situation, where flood defences have been part of the countries critical infrastructure for centuries. Different countries may have different protocols for emergency measures, leading to different reliabilities of the flood defence system. Even though these differences may lead to different reliabilities, the method developed in this chapter can be applied in other flood prone areas protected by flood defence systems, to determine the combined reliability of the system.

4

Optimizing portfolios of risk reduction strategies: flood defences and/or land fills

Throughout history, mankind has tended to settle along low-lying coastlines and river deltas, because of the advantages for agriculture and trade, despite the risk of flooding. Protection of flood prone areas can be provided by different types of measures. Examples include raising land, raising structures, floodproofing structures, constructing permanent or temporary flood defences.

This chapter builds on existing methods for optimizing the total cost of flood risk reduction strategies by introducing fills, including the variability of the size of the protected area, its land use and corresponding value. The method is used to minimize the total costs of flood risk reduction for several practical examples to determine which strategy is preferred from an economical point of view. In doing so, this chapter aims to clarify why different protection strategies have been chosen in practice and what may have driven these decisions. Ultimately, helping decision makers to decide between different strategies for future decision making.

This chapter is in preparation for submission to the Journal of Flood Risk Management.

4.1 Introduction

With rising sea levels, and studies that claim flood events are increasing both in magnitude and in frequency (EEA 2012; R. S. Kovats et al. 2014), the need for effective flood risk reduction strategies is becoming more and more important. This is the case for existing flood-prone areas, but also for areas yet to be developed, such as land reclamations and new developments in flood prone areas.

Different strategies can be applied to reduce flood risks. One strategy is to surround an area subject to flood risk with flood defences, creating a 'polder'. A drainage system is installed to drain excess water from the polder to the adjacent rivers or sea. The technique worked so well that the Dutch decided to invest a large part of the profits of the golden age during the 17th century in reclaiming polders like the Beemster and the Heerhugowaard in the province of North Holland. Currently, a large part of the Netherlands consists of polders. The last major polder reclamation project in the Netherlands was the Flevopolder, which was finished in 1968. Polders were also built in countries such as Germany (along the Weser and Elbe rivers), England (along the Fens near Boston), Surinam, Bangladesh and India, where large polders were built in marshes for agriculture purposes. More recent examples include the airport of Suvarnabhumi in Thailand (Seah 2005), which remained operational during the floods of 2011 while the Don Muong airport in Thailand was completely flooded (ENW 2012). Finally, the new Pulau Tekong development in Singapore is also built in a polder.

Another strategy for reducing flood risks is to limit potential flood consequences by raising or flood proofing of structures, or raising entire areas well above expected flood levels, creating large elevated land fills (or mounds). Local examples of this strategy can be found along low-lying coastal areas in the United States (e.g., Bolivar Peninsula, Texas), where locals raise houses on piles (Tomiczek et al. 2013), or in unembanked areas in the Netherlands, where farmers build houses on top of large soil mounds. Larger scale examples are also found: in Singapore, land reclamations have almost exclusively been built on large fills; as are the airport of Hong Kong and several port expansion projects in Jakarta (IPC port developer 2012). Massive land fills were also used for large reclamation projects in the Netherlands: examples include the Botlek, Europoort and the 2nd Maasvlakte areas in the port of Rotterdam and the IJburg reclamation project in Amsterdam (de Leeuw et al. 2002). Combinations of interventions can also be found, such as the coast of Japan, where flood defences are combined with land fills and large sea walls to mitigate tsunami risks after the 2011 Tohoku Tsunami (Strusińska-Correia 2017).

While both strategies have pros and cons, a key question is which strategy is optimal for a given situation. Land fills generally reduce damages but require large volumes of soil (or structural interventions) when applied to large scale areas. The required soil volumes for flood defences are generally smaller, but consequences in case of failure of the defences will be larger than for the land fill strategy. Economic optimization methods are often applied within different fields of civil engineering: examples include tunnels (Arends et al. 2005), coastal and port infrastructure (Mai et al. 2009; Nagao et al. 2003). Specifically in flood risk management, economic optimization has been used to determine optimal elevation levels of flood defences and land fills (Tsimopoulou et al. 2014; Vrijling 2014; Kind 2014). Related optimization models have also been used to assess the impact of flood proofing measures on residential building vulnerability (Custer 2015), to support decisions about the implementation of flood mitigation measures at different points in time (Woodward et al. 2014) or to assess the trade-off between levee setback or heightening (Zhu & Lund 2009).

A number of limitations hamper the ability of existing methods to investigate trade-offs between flood defences and mitigation of consequences (e.g. by land fills). Most approaches in this field only focus on one intervention, mostly flood defences (Kind 2014; Vrijling 2001; Eijgenraam 2006). Approaches that consider both defences and land fills (Tsimopoulou et al. 2014) approach this problem numerically and for a limited number of alternatives, without an analytical solution. Existing approaches also mostly assume a specific case or area, which does not lead to more generally applicable insights for different area sizes.

Solving the optimization model analytically can provide insight in a) the largest drivers of cost and risk and b) the influence of different variables (e.g. size and value of the area, costs of interventions and potential damages) on the preferred flood risk reduction strategy. The aim of this work is to develop a (largely) analytical solution for optimizing the trade-off between flood defences and land fills, and the corresponding optimal elevation levels. The costs, damages and risks, and properties of the considered area are taken into account. The model will include the dependency of damages on flood depth, to accurately model flood risk and particularly the effect of raising of structures by means of land fills or other measures.

Overall, the model will aid in better understanding of the cost and risk associated with flood defences and land fills, and the drivers that influence these costs. By doing so, this work aims to clarify which risk reduction strategy is optimal in a given condition, and to give insight in what drives these decisions. For this purpose, a mathematical model is built that enables economic optimization of both strategies and their combinations. Several practical examples are discussed to analyze situations where a strategy containing one or multiple layers of protection (i.e., a land fill and/or a flood defence) is preferred from an economical point of view.

Besides flood defences, this chapter focuses on land fills (hereafter 'fills'), since this intervention is representative for approaches to reduce the economic consequences of floods, such as flood proofing or raising structures. In addition, the chapter focuses on economic engineering considerations, effects of strategies on risk to life are not explicitly considered. Other flood risk reduction strategies such as land use planning, insurance, evacuation and emergency measures for flood prevention are also beyond the direct scope of this chapter.

The chapter is organized as follows: Section 4.2 derives the methodology that enables optimization of the costs of raising land and constructing flood defences (or a combination of both), depending on the area to be developed and its value. Section 4.3 discusses what type of protection is preferred from an economical point of view for specific practical examples. Section 4.4 discusses other advantages and disadvantages associated with both protection strategies that may influence decision making. Finally, Section 4.5 concludes with the main findings of this chapter and directions for future research.

4.2 Methodology

This section derives a mathematical model that enables optimization of the costs associated with raising land or constructing flood defences to protect a specific area from flooding. The mathematical model builds on existing models used to evaluate the economic implications of multi-layered safety (Tsimopoulou et al. 2014; Vrijling 2014; Jonkman et al. 2005) and to optimize land reclamation levels for port terminals (Lendering et al. 2015). Generally, these models compare the total costs of flood protection based on a summation of the investment cost and the present value of flood risk (i.e., the expected damages due to flooding over the lifetime), see Eq.1. Flood risk is defined here as the expected annual damages of flooding. This approach was first used by van Dantzig to optimize elevation levels for flood defences in the Netherlands (van Dantzig 1956); his method was later improved by Eijgenraam (2006) to account for time dependencies (e.g., economic growth, degradation of flood defences and sea level rise) (see also (Kind 2014)).

$$\textit{Total cost} = \textit{Investment} + \textit{Risk} \tag{4.1}$$

The total costs are minimized to determine the optimal elevation level (e.g., the flood defence or fill level), or in other words, what elevation level minimizes the cost. Here, for simplicity, time dependencies of optimal safety levels are neglected. The following subsections first derive or compare the investment costs (4.2.1) and present value of the risk (4.2.2) associated with raising land or flood defences. The total cost for both options are found by summing the investment cost and present value of the risk. These total costs are then minimized to obtain the optimal elevation levels for different practical cases and increasing size of the area to be developed (4.2.3).

4.2.1 Investment cost

The investment cost of the considered flood risk reduction strategies depends on the size of the area to be protected, the elevation level and the marginal cost of each option. We consider an area where a housing project is planned on a floodplain see Figure 35. The base level of the area is modelled by h_0. The area to be protected from flooding is modelled by variable *A* and depends on the land use (e.g., other land uses include agriculture or industry), the amount of structures and their footprint (e.g., 200m^2 per structure). Additional space is reserved for infrastructure in and around the area. The total size of the area to be developed is found by Eq. 4.2:

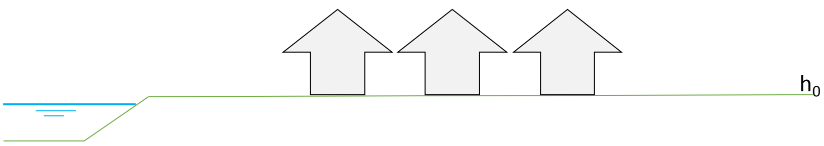

Figure 35: Development on a number of houses that require flood protection

$A = N_d \cdot A_d \cdot C_d$ $[m^2]$ (4.2)

with N_d = the number of structures (-), A_d = the footprint per structure (m^2) and C_d = an addition for infrastructure around each development (-)

The investment cost function is derived for both fills and flood defences, see Figure 36. The investment cost of a fill is a function of the area *(A)*, the crest level *(h_m)* and the marginal cost for raising land *(C_m)*, see Eq. 4.3.

$I_m = A \cdot (h_m - h_0) \cdot C_m$ [€] (4.3)

with A = area of the fill, h_m = crest level of the fill and C_m = unit cost to build the fill [€/m/m²]

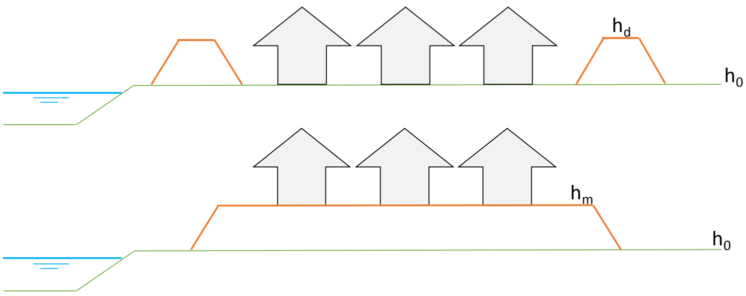

Figure 36: Surrounding the area with flood defences (top) and raising a large fill (bottom) are considered as options to provide flood protection

A similar relation is derived to estimate the investment cost of a system of flood defences surrounding the area. We assume that the total area to be protected is circular. The length of the flood defences that surround the area is derived with Eq. 4.4, which calculates the circumference of the area. Ultimately, the investment costs of the flood defence system *(I_p)* are a function of the marginal cost for a flood defence *(C_d)*, its length *(L)* and the crest level *(h_d)*, see Eq. 4.5.

$L = 2 \cdot \sqrt{\pi \cdot A}$ [m] *(i.e., the circumference of a circular area)* (4.4)

$I_p = L \cdot (h_d - h_0) \cdot C_d = 2 \cdot \sqrt{\pi \cdot A} \cdot (h_d - h_0) \cdot Cd$ [€] (4.5)

with L = length [m], h_d = crest level of the flood defence (or dike) [m] and *C_d = marginal cost for the flood defence [€/m/m]*

The investment cost of a fill increases linearly with the size of the area, while the investment cost of the fill has a quadratic relation with the area, as illustrated in Figure 37. In general, for small areas, the investment cost for raising land is lower than the cost of flood defences. For larger areas, flood defences result in lower costs.

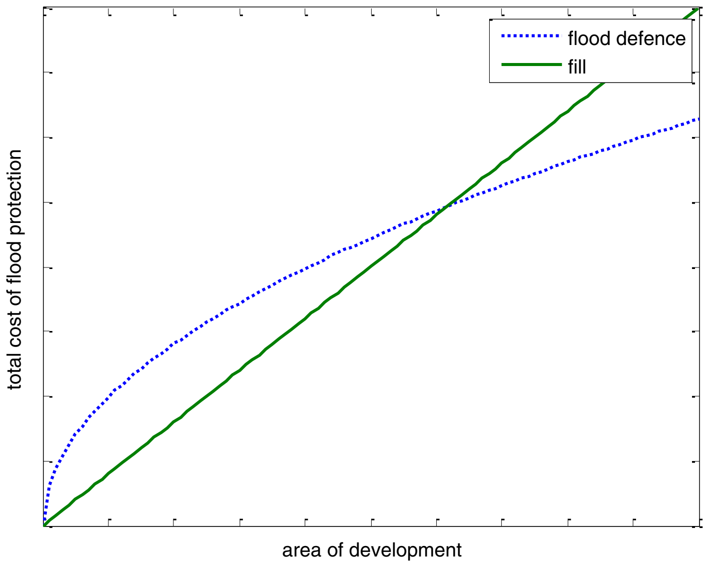

Figure 37: Investment cost for a fill (blue dotted line) and a system of flood defences (red straight line), depending on the area that requires protection

The area where the investment cost of a system of flood defences becomes lower than those of raising land is modelled with variable A_t, is found with Eq.6:

$$A_t = 4 \cdot \pi \cdot \left[\frac{C_d \cdot (h_d - h_0)}{C_m \cdot (h_m - h_0)}\right]^2 \quad [m^2] \qquad (4.6)$$

Note that a straightforward prioritization of flood defences over fills, based on a solely the comparison of their respective investment cost, is not complete, because this ignores the differences in flood risk. These topics are discussed in the following subsections.

4.2.2 Risk of flooding

Risk is estimated by the present value of flood risk over the considered lifetime (i.e., the expected value of damages). It is defined by the product of the annual probability of flooding with its potential consequences; as shown in Eq. 4.7. The annual probability of flooding depends on the considered flood risk reduction system (if any) and the potential flood hazards (e.g., fluvial, pluvial or coastal flooding). For a fill, the probability of flooding is determined by the probability that water levels exceed the crest level of the fill. For an area surrounded by flood defences, the probability of flooding is determined by the probability that these flood defences fail (i.e., breach). Failure may occur due to overflow or geotechnical failure (e.g., due to piping or instability). In our model, the flood probability of an area protected by flood defences is estimated by the probability of overflowing. It is then assumed that the probability of other failure mechanisms is negligibly small compared to the probability of overflowing. Design guidelines such as (Ciria 2014) provide guidance for the design of flood defences with negligible probabilities of failure mechanisms other than overflow.

The risk is quantified with Eq. 4.8, given a probability density function of annual water levels *f(h)* and a damage function $D(h_w)$. The present value of flood risk is determined with Eq. 4.8 (i.e. the present value of expected damages) for a finite lifetime *(T).* For simplicity, the lifetime is assumed to be infinite, which simplifies Eq. 4.8 in to Eq. 4.9. The probability density function of water levels and the damage function are derived in the following subsections.

$$R = \int f(h_w) \cdot D(h_w)\, dh_w \ [€/yr] \quad (4.7)$$

$$R = \sum_{t=1}^{T} \frac{\int f(h_w) \cdot D(h_w) dh_w}{(1+r)^t} \quad (4.8)$$

$$R = \frac{\int f(h_w) \cdot D(h_w) dh_w}{r} \ for\ T \rightarrow \infty \ [€] \quad (4.9)$$

Probability density function of water levels

The annual probability of flooding is estimated by the probability of water levels exceeding the crest level of the fill (h_m) or flood defence (h_d), as illustrated in Figure 38. Annual extreme water levels (h_w) are typically described by an exponential distribution (van Dantzig 1956; Jonkman et al. 2005) with constants a, the *location* parameter in meters, and b, the *scale* parameter in meters (see Eq.10).

$$\mathrm{f(h)} = \frac{e^{-\frac{h_w - a}{b}}}{b} \quad (4.10)$$

Eq.11 describes the probability of non-exceedance of water levels ($P(h_w \leq h_d$ *or* $h_m)$).The probability of exceedance of water levels $(P(h_w > h_d\ or\ h_m))$, which represents the probability of overflow is found with Eq.12:

$$F(h) = 1 - e^{-\frac{h_w - a}{b}} \ [yr^{-1}] \quad (4.11)$$

$$P_f(h_w > h_d, h_m) = 1 - F(h) = e^{-\frac{h_w - a}{b}} \ [yr^{-1}] \quad (4.12)$$

Damage function

The damage of flooding is typically divided in direct damages (i.e., material damages) and indirect damages (i.e., business losses). For simplicity and in line with the approach proposed by Hallegatte (2013) indirect damages are assumed to increase linearly with direct damages. Here, they are included in the (direct) damages, and depend on the land use (e.g., housing, agriculture or industry).

The direct damage of flooding is determined by the value of the protected area, which, among others, consists of the value of all structures and infrastructure in the area. The potential direct damages (D_{pot}) are quantified with Eq.13, which is a function of the size *(A)* and value *(V)* of the area.

$$D_{pot} = A \cdot V \ [€] \quad (4.13)$$

A linear relation is used to estimate flood damages for increasing flood depths. In case of dike failure, water levels in the protected area are assumed to become equal to the outside water level. Other flood characteristics and dynamics (flow velocity, rise rate) are not included in the simplified modelling here. Thus, for an area behind a flood defence, the flood depth after dike failure is equal to the water level *(h_w)* minus the initial level of the floodplain *(h_w - h_0)*. For fills, the flood depth is equal to water level minus the level of the fill *(h_w - h_m)*. This is illustrated by the following figure:

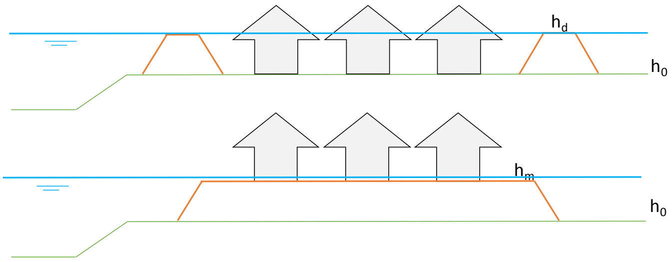

Figure 38: Flooding of an area in a polder and on a fill

Damage will only occur when the water levels exceed the crest of the flood defence or fill. Flood damages are bounded by a maximum food depth *(d_{max})*, for which all value is assumed to be lost, which depends on the land use. For example, agricultural land will have a lower value of d_{max} than industry or housing, because all crops are assumed to be lost once the surface of the land is flooded. The resulting damage functions for fills (Eq. 14) and flood defences (Eq. 15) are included below:

$$\begin{aligned} D_{fill} &= 0 && [€] && h_w < h_m \\ D_{fill} &= D_{pot} \cdot \frac{h_w - h_m}{d_{max}} && [€] && h_m < h_w < h_m + d_{max} \\ D_{fill} &= D_{pot} && [€] && h_w > h_m + d_{max} \end{aligned} \tag{4.14}$$

$$\begin{aligned} D_{dike} &= 0 && [€] && h_w < h_d \\ D_{dike} &= D_{pot} \cdot \frac{h_w - h_0}{d_{max}} && [€] && h_d < h_w < h_0 + d_{max} \\ D_{dike} &= D_{pot} && [€] && h_w \geq h_0 + d_{max} \end{aligned} \tag{4.15}$$

The relations between flood damage and flood level are illustrated in the conceptual damage functions in the left graph of Figure 39, which assumes that the flood defence level is lower than the level at which maximum flood damages occur ($h_d < h_0 + h_{max}$). When the flood defence level is higher than the level at which maximum damages occur ($h_d > h_0 + d_{max}$), all value inside the flood defence system will be lost once the flood defence level is exceeded. This is illustrated in the right graph of Figure 39 (and depicted in Eq. 16), where flood damages are maximum once the water level exceeds the flood defence level. The graphs also illustrate that, for a given water level, the flood damages of an area protected by flood defences (i.e., a polder) are higher than the damages to the same area on top of a fill.

$$\begin{aligned} D_{dike} &= 0 && [€] && h_w < h_d \\ D_{dike} &= D_{pot} && [€] && h_w \geq h_d \end{aligned} \tag{4.16}$$

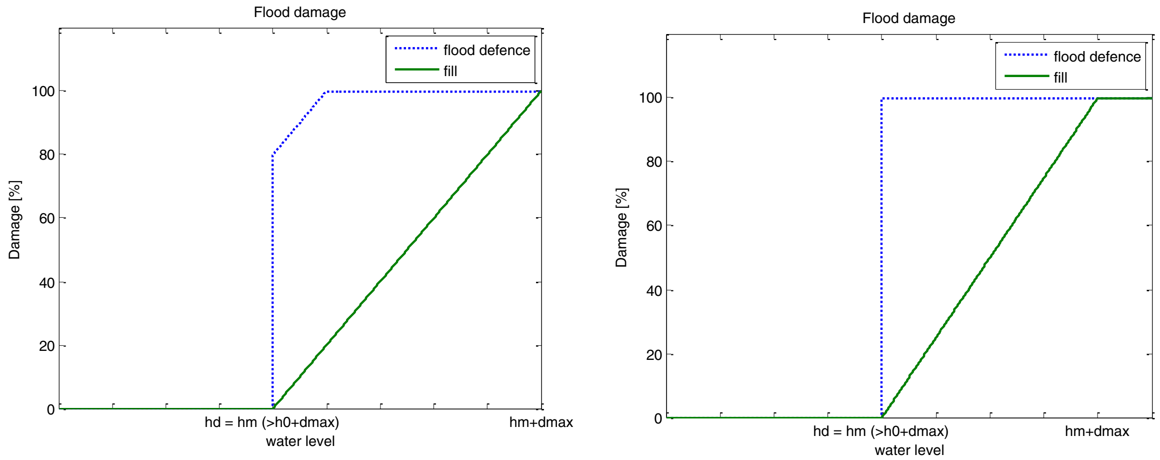

Figure 39: Damage function for $h_d < h_0+d_{max}$ (left) and for $h_d > h_0+d_{max}$ (right)

Risk is found by multiplying the potential food damages with the probability of flooding. The following conceptual graph illustrates the present value of the risk for increased elevation levels of fills and flood defences. The risk decreases with increased elevation levels for both strategies, because the probability of flooding reduces with increased elevation levels. Notice how the risk of flood defences is always higher than of fills, because once an elevation level is exceeded, polders fill completely resulting in higher damages than fills, because of the limited flood depth on the fill. The optimization of flood risk against the investment cost associated with raising land or constructing flood defences is explained in the following subsection.

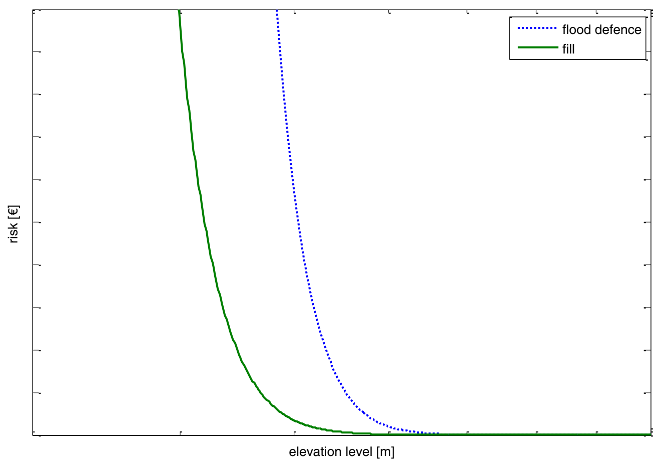

Figure 40: Conceptual graph of flood risk depending on the elevation level of fills or flood defences

4.2.3 Total Costs and Optimization

Economical optimization is used to determine the optimal elevation levels for raising flood defences and fills, based on the total costs of both strategies. The total costs are found by summing the investment costs with the present value of the risk (Eq. 4.17). The minimal total costs are determined by minimizing the respective total cost functions with respect to the fill and flood defence level (Eq. 4.18). The following subsections derive the equations for the optimal fill and optimal flood defence level.

$$TC\ (h_{d;m}) = I(h_{d;m}) + R\ (h_{d;m}) \tag{4.17}$$

$$\frac{\partial TC}{\partial h_{d;m}} = 0 \tag{4.18}$$

with TC = Total Cost, I = Investment cost and R = present value of Risk.

Optimization of total costs of a fill

The total cost function for a flood risk reduction system consisting of a fill *(TC_m)* is found by combining Equations 4.1, 4.3, 4.9, 4.10, and 4.14, providing the following function:

$$TC_m = A \cdot (h_m - h_0) \cdot C_m + R_m$$

$$\text{with} \quad R_m = \frac{\int \frac{e^{-\frac{h_w - a}{b}}}{b} \cdot A \cdot V \cdot \frac{h_w - h_m}{d_{max}} \cdot dh_w}{r} \qquad \text{for } h_m < h_w < h_m + d_{max}$$

$$R_m = \frac{\int \frac{e^{-\frac{h_w - a}{b}}}{b} \cdot A \cdot V\ dh_w}{r} \qquad \text{for } h_w > h_m + d_{max} \tag{4.19}$$

After solving the integral in the risk function, the following function is found (detailed derivations are provided in the appendices):

$$R_m = \frac{A \cdot V \cdot b}{r \cdot d_{max}} \cdot \left(e^{-\frac{h_m - a}{b}} - e^{-\frac{h_m + d_{max} - a}{b}}\right) \tag{4.20}$$

Including the risk function in the total cost function gives the following equation for total costs of a fill:

$$TC_m = A \cdot (h_m - h_0) \cdot C_m + \frac{A \cdot V \cdot b}{r \cdot d_{max}} \cdot \left(e^{-\frac{h_m - a}{b}} - e^{-\frac{h_m + d_{max} - a}{b}}\right) \tag{4.21}$$

Note that the present value of the risk of floodplains without flood risk reduction measuresis found when solving Eq. 4.21 for $h_m = h_0$. Eq. 4.21 is minimized to obtain the optimal level of the fill *($h_{m;optimal}$)*,

$$\frac{\partial TC}{\partial h_m} = A \cdot C_m + \frac{A \cdot V}{r \cdot d_{max}} \cdot \left(e^{-\frac{h_m + d_{max} - a}{b}} - e^{-\frac{h_m - a}{b}}\right) = 0$$

$$h_{m;optimal} = a - b \cdot \ln \frac{c_m \cdot d_{max} \cdot r}{V \cdot (1 - e^{-\frac{d_{max}}{b}})} \tag{4.22}$$

The optimal elevation level of a fill depends on the parameters (a and b) of the exponential distribution of water levels, the marginal cost for raising fills *(C_m)*, the depth where flood damages are maximized *(d_{max})* and the marginal value of the area *(V)*. It is not influenced by the size of the area, since both the cost and damages increase linearly with the area.

Optimization of total costs of a polder

The total cost function for a system of flood defences surrounding a (circular) polder (TC_d) is found by combining Equations 4.1, 4.5, 4.9, 4.10, 4.13 and 4.15, providing the following function:

$$TC_d = L \cdot (h_d - h_0) \cdot 2 \cdot \sqrt{\pi \cdot A} \cdot Cd \quad + R_d \tag{4.23}$$

with $R_d = \dfrac{\int \frac{e^{-\frac{h_w-a}{b}}}{b} \cdot A \cdot V \cdot \frac{h_w-h_0}{d_{max}} \cdot dh_w}{r}$ *for* $h_d < h_w < h_0+d_{max}$

$R_d = \dfrac{\int \frac{e^{-\frac{h_w-a}{b}}}{b} \cdot A \cdot V \, dh_w}{r}$ *for* $h_w > h_0+d_{max}$

After solving the integral in the risk function, the following function is found (detailed derivations are provided in the appendix):

$$R_d = \frac{A \cdot V \cdot b}{r \cdot d_{max}} \cdot \left(\left[\frac{h_d}{b} - \frac{h_0}{b} + 1 \right] e^{-\frac{h_d-a}{b}} - e^{-\frac{h_0+d_{max}-a}{b}} \right) \tag{4.24}$$

Including the risk function in the total cost function provides the following equation for total costs of a flood defence system, surrounding a polder:

$$TC_d = (h_d - h_0) \cdot 2 \cdot \sqrt{\pi \cdot A} \cdot C_d + \frac{A \cdot V \cdot b}{r \cdot d_{max}} \cdot \left(\left[\frac{h_d}{b} - \frac{h_0}{b} + 1 \right] e^{-\frac{h_d-a}{b}} - e^{-\frac{h_0+d_{max}-a}{b}} \right) \tag{4.25}$$

Eq. 4.25 is minimized to obtain the optimal elevation level of the flood defence *($h_{d;optimal}$)*:

$$\frac{\partial TC}{\partial h_d} = 2 \cdot \sqrt{\pi \cdot A} \cdot C_d + \frac{A \cdot V}{r \cdot d_{max}} \cdot \left(\left[\frac{h_0}{b} - \frac{h_d}{b} \right] \cdot e^{-\frac{h_d-a}{b}} \right) = 0$$

$$(h_d - h_0) \cdot e^{-\frac{h_d-a}{b}} = \frac{2 \cdot \sqrt{\pi \cdot A} \cdot C_d \cdot d_{max} \cdot r \cdot b}{A \cdot V} \tag{4.26}$$

Eq. 4.26 can be rewritten in the form W· exp(W) = Z, in which Z is a given constant and W the unknown variable. This so-called 'Lambert function' is solved numerically in the practical examples in Section 4.3, because functions of this type cannot be solved analytically. Other examples with similar functions are found in (Kok et al. 2002; Lendering et al. 2014). The optimal flood defence level depends on the parameters (a and b) of the exponential distribution of water levels, the marginal cost for raising flood defences *(C_d)*, the depth where flood damages are maximized *(d_{max})* and the marginal value of the area to be protected *(V).* In addition, the initial level of the area (h_0) to be protected and the size of the area to be protected *(A)* also influence optimal flood defence levels. As will be shown in Section 4.3, larger areas to be protected result in higher optimal elevation levels for flood defences.

Note that the previous derivations (Eq. 4.23 to Eq. 4.26) represent situations where the flood defence level is lower than the water level where maximum damages occur

$(h_d < h_0+d_{max})$. If the flood defence level exceeds the level where maximum damages occur $(h_d > h_0+d_{max})$, all behind the flood defence is lost once it fails. This assumption is often used in previous work (Vrijling 2001; van Dantzig & Kriens 1960). In that case, Eq 4.25 simplifies to Eq. 4.27 and the optimal flood defence level is found with Eq. 4.28.

$$TC_d = (h_d - h_0) \cdot 2 \cdot \sqrt{\pi \cdot A} \cdot Cd + \frac{A \cdot V}{r} \cdot e^{-\frac{h_d - a}{b}} \qquad \text{for } h_d > h_0+d_{max} \qquad (4.27)$$

$$h_{d;optimal} = a - b \cdot \ln\left[\frac{2 \cdot \sqrt{\pi \cdot A} \cdot Cd \cdot b \cdot r}{A \cdot V}\right] \qquad \text{for } h_d > h_0+d_{max} \qquad (4.28)$$

The following conceptual graph illustrates the investment, risk and total costs of fills and flood defences, given optimal elevation levels and increasing size of the protected area (Figure 41). The example is based on the values included in Table 17. For small areas, the total costs of fills are lower than the total cost of flood defences.

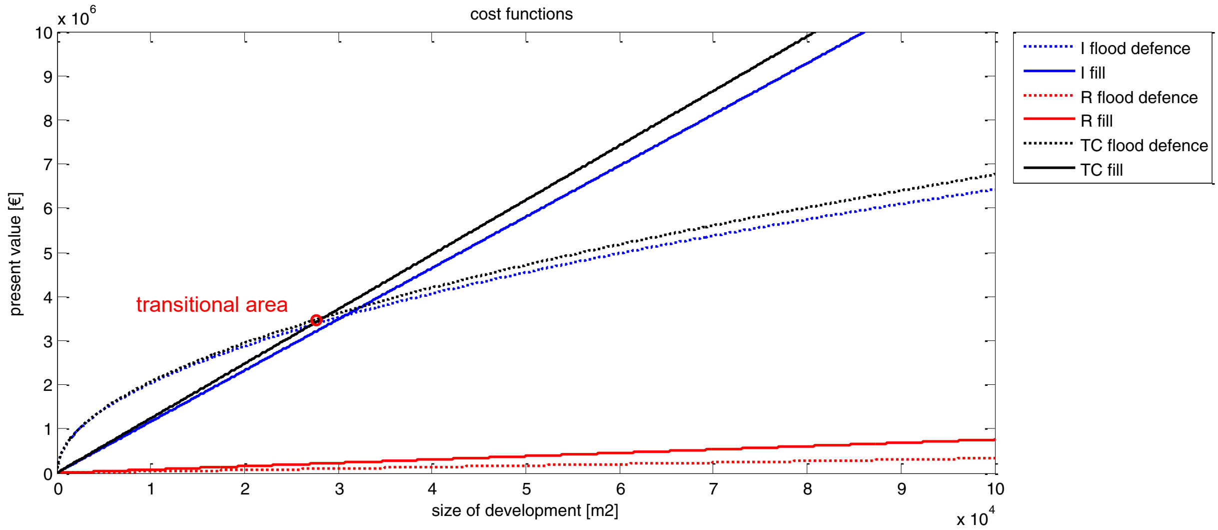

Figure 41: Investment, risk and total costs for increasing size of the protected area

The transitional area is defined as the area size for which the total cost of fills and flood defences are equal, see Figure 41. The following graphs illustrate the investment cost, risk and total costs of both flood defences and fills for areas smaller (Figure 42) and larger (Figure 43) than the so-called transitional area (given the data in Table 17). For areas smaller than the transitional area, fills are always preferred over flood defences since this strategy will lead to lower total costs. For areas larger than the transitional area, flood defences are preferred over fills from a certain elevation level: the transitional elevation level. This level is found by finding the elevation level for which the total cost of both strategies is equal. A similar analysis was performed in (Lendering et al. 2015), where elevation levels for land reclamation where optimized for fills and polders.

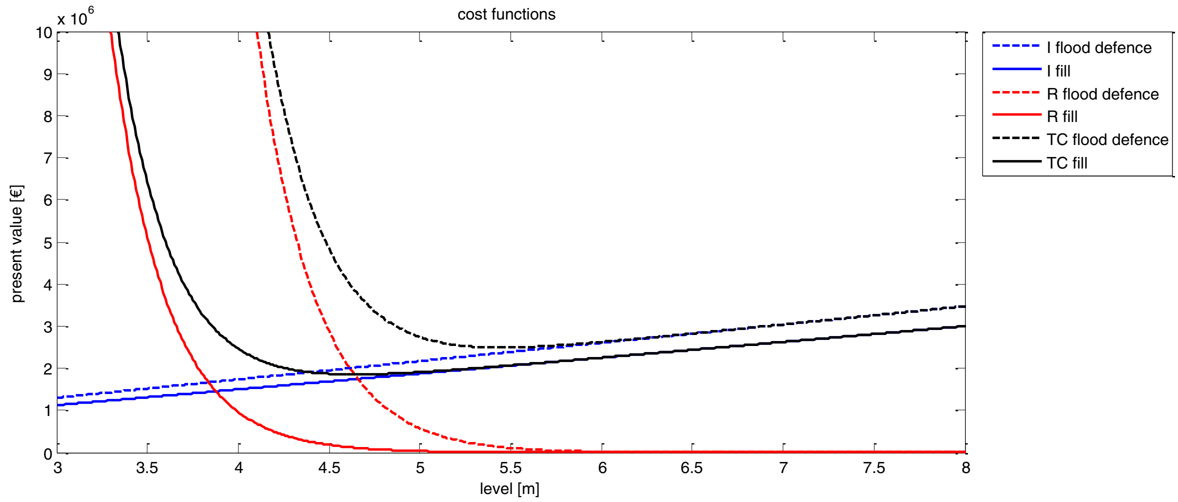

Figure 42: Investment, risk and total cost for optimal elevation levels, given an area smaller than the transitional area. Here, the fill is preferred over flood defences.

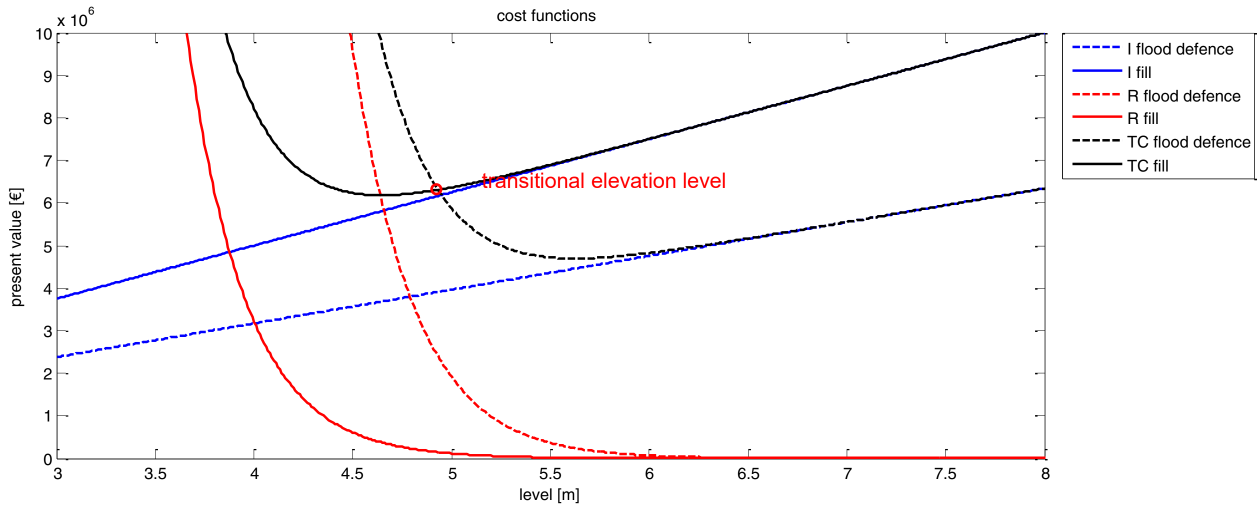

Figure 43: Investment, risk and total cost for optimal elevation levels, given an area larger than the transitional area. Here, flood defences are preferred over fills.

4.3 Cases

4.3.1 Introduction

The mathematical relations derived in the previous section are used to analyse the costs and risks associated with both flood risk reduction strategies and their combination. While the examples in the chapter have been highly simplified, they do represent realistic, practical cases in which decision makers are faced with deciding between flood protection or consequence mitigation. Table 17 contains the generic input variables used in the examples.

Description	Variable	Value
The base level of the floodplain.	h0	0 m
Location (a) and scale (b) parameters of the exponential water level distribution. This level represents a case along the North Sea, with a relatively mild sloping exponential distribution.	a b	3 m 0.3 m

Discount factor for discounting risk over the lifetime.	r	0.05
Marginal cost for raising land with a fill (including cost of outer slope protection against erosion).	C_m	25 €/m/m^2
Marginal cost for constructing flood defences (5 million euro per kilometer for levees with a retaining height of 5 meters (Jonkman et al. 2013)).	C_d	1,000 €/m/m
Value of residential/ industrial land and depth for maximum damages.	V	1,500 €/m^2
	d_{max}	5 m
Value of agricultural land and depth for maximum damages.	V	100 €/m^2
	d_{max}	1 m

Table 17: Generic case study variables

This section is divided in three subsections: Subsection 4.3.1 discusses examples where an area inside a floodplain is developed for a single land use, while also analysing the sensitivity of the results to different marginal cost and land use values. Subsection 4.3.2 discusses examples where an area on a floodplain is developed for multiple land uses and Subsection 4.3.3 discusses optimal risk reduction strategies given the presence of an existing flood defence. Finally, Subsection 4.3.4 discusses the costs associated with both options if a minimum safety level is required.

4.3.1 Single land use

We consider a residential area to be built on a flat floodplain along a river or coast. Flood risk reduction can be provided by raising the entire residential area to a level well above flood levels or by surrounding the area with a (circular) system of flood defences (see Figure 36). The optimal levels for fills and flood defences with increasing size of the protected area are determined, based on the values included in Table 17. The results are shown in the following graphs:

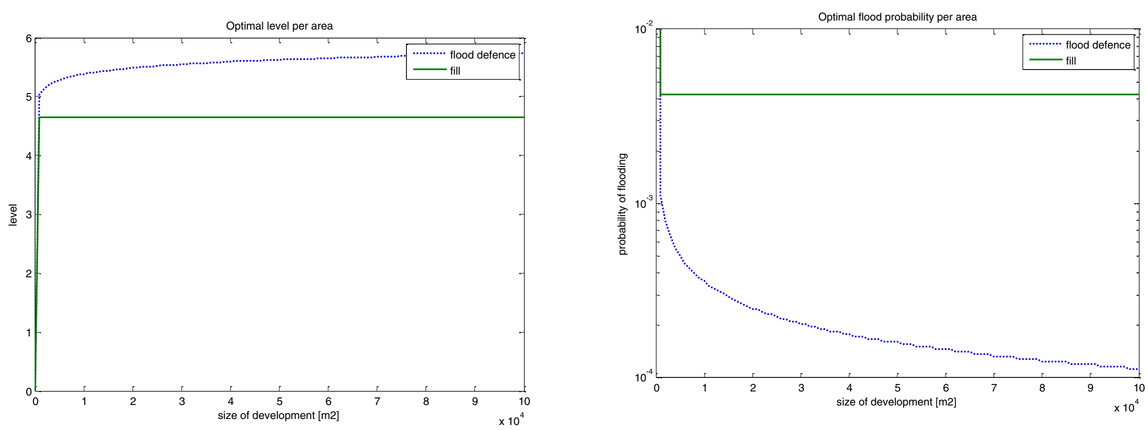

Figure 44: Optimal level (left) and flood probabilities (right) of raising the residential area on a fill or surrounding the area with flood defences, for increasing size of the protected area.

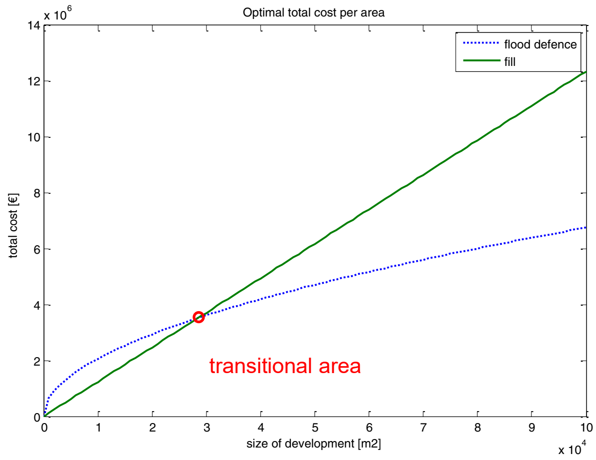

Figure 45: Total costs for optimal elevation levels of a fill and a system of flood defences, given increasing size of the protected area.

We find that the optimal level for fills is a constant, irrespective of the area that is protected (the optimal level is not a function of the area, see Eq. 4.22). The optimal level for flood defences increases with increasing size of the area, as the optimal level is a function of the area (Eq. 4.28). Note that in this example the optimal elevation level of the fill is lower than that of a system of flood defences, because the optimal flood probabilities for fills are larger than for flood defences.

Figure 45 illustrates the resulting total costs for optimal elevation levels of both a fill and a system of flood defences. Raising fills is only beneficial (economically) if the total area is smaller than the transitional area size (in the example about 30,000 square meters or 2,000 single layer houses with an average footprint of 150 square meters). A system of flood defences is more economical over raising the area on top of a fill, provided that the area is larger than the transitional area and optimal elevation levels are chosen.

Sensitivity to the marginal costs

The previous example assumed a given constant ratio between the marginal cost for fills and flood defences. The following subsection discusses the sensitivity of the results to this ratio. In this analysis, the optimal levels, probabilities and total costs for significantly higher and lower marginal costs of both fills and flood defences is estimated.

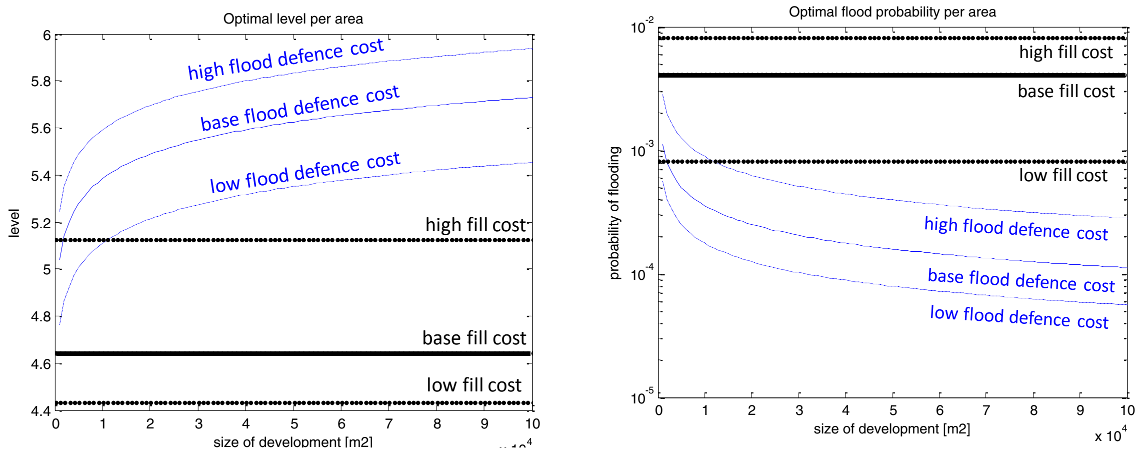

Figure 46: Sensitivity of the optimal elevation level (left) and probability of flooding (right) to the marginal costs. The solid blue and black lines represent the base case of a flood defence and fill respectively. The dotted lines represent the scenario with low and high costs.

The solid blue lines represent the base case of flood defences (1,000 €/m/m), while the dotted blue lines are the optimal levels for marginal costs of 2,500 €/m/m and 500€/m/m. The solid black lines represent the base case of fills (25 €/m/m^2), while the dotted black lines are the optimal levels for marginal costs of 5 €/m/m^2 and 50€/m/m^2. The optimal level of both fills and flood defences reduce with increasing marginal cost, and vice versa for decreasing marginal cost. In summary, higher costs for raising fills or flood defences result in lower optimal elevation levels.

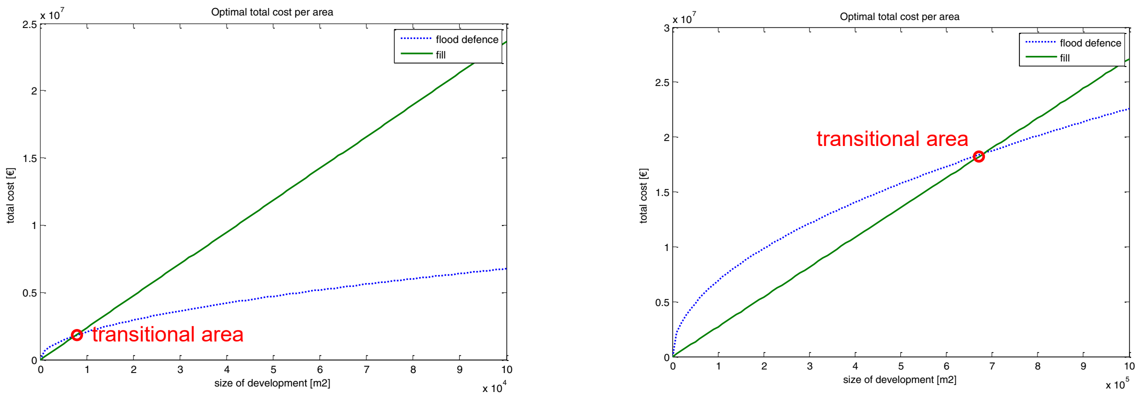

Figure 47: Total costs for optimal elevation levels of a fill and a system of flood defences, given increasing size of the residential area.

The increased marginal cost of fills (50 €/m/m^2) benefit flood defences over fills (left graph of Figure 47). For this specific example, flood defences are more economical for areas larger than about 7,000 square meters. Such high costs for fills may represent projects where existing residential areas are to be raised or redeveloped to reduce flood risk. In contrast, reduced marginal cost for fills (5 €/m/m2) result in a larger fill development size where fills are preferred over flood defences, as shown in the right graph of Figure 47.

Sensitivity to the land use value

The sensitivity of the model to different land use values is analyzed by reducing its value significantly, which may represent agricultural land use (the values included in Table 17 represent a residential or industrial area).

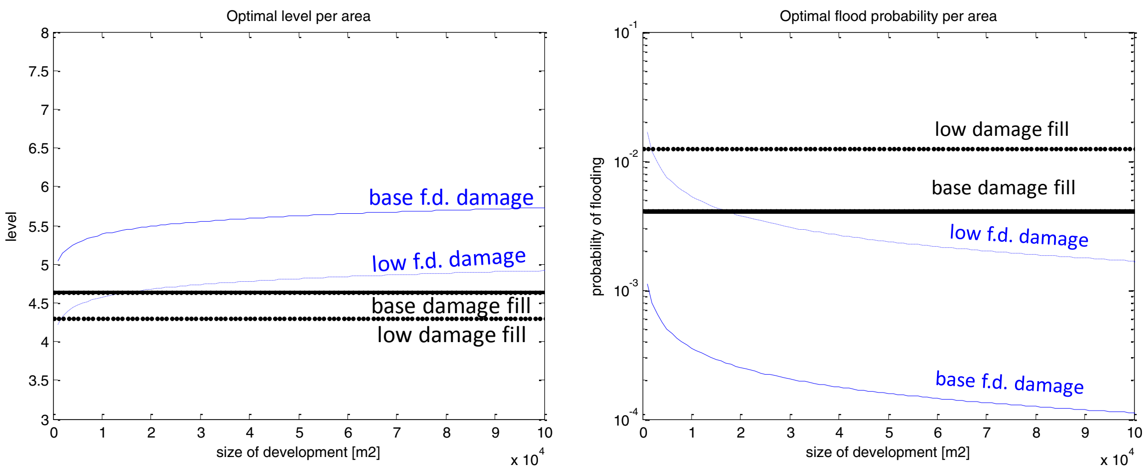

Figure 48: Optimal level (left) and flood probabilities (right) of raising a fill or surrounding the area with flood defences (f.d.), for increasing size of the agricultural area. The solid lines represent the base case residential area, with damages of 1,500 €/m^2.

Smaller optimal elevation levels for both fills and flood defences are found due to a reduction of the land use value. The transitional area (30,500 square meters) does not change significantly compared to the base case (Figure 45), suggesting that the sensitivity of the transitional area to the land use value is small.

Results

The results for the base case are based on typical values for western countries such as the Netherlands, with relatively high investment costs and damages. While the potential damages do not have a large influence on the preferred strategy (only on optimal elevation levels), high marginal costs result in a strong preference for flood defences over fills. In contrast, low costs and damages (e.g., for developing countries) would typically lead to a stronger preference for fills, because the area where flood defences become more economical over fills increases significantly.

4.3.2 Combining multiple land uses

This subsection discusses examples in which multiple types of land use are combined in an area, each possibly requiring a different strategy for risk reduction. For example, combining agricultural land with a residential area, as illustrated in Figure 49. Note that a flood defence in front of a fill will lower the flooding probability of that fill if it has a higher level than the fill (and vice versa, because the flood defence and fill are dependent and correlated through the water level). The flood defence will then reduce the probability that the fill floods, thus also reducing the risk associated with the protected area on the fill.

Based on the findings of the preceding section, we find that a flood defence is economically more beneficial than a fill for areas larger than the transitional area. In the example, the transitional area for residential areas was about 30,000 square metres, while the same transitional area for agricultural land use was about 30,500 square meters. Different combinations of land uses and preferred flood risk reduction strategies are shown in Table 18 and Figure 49.

	Agriculture area smaller than transitional area	**Agriculture area larger than transitional area**
Residential area smaller than transitional area	Both agriculture and residential area on fills	Combination preferred: agriculture behind (small) flood defence and residential area on fill
Residential area larger than transitional area	Combination preferred: agriculture on fill and residential area behind flood defence, although possibly more practical to have both land uses be protected by a flood defence.	Both agriculture and residential area behind flood defence

Table 18: Combination of multiple land uses and risk reduction strategies in a floodplain

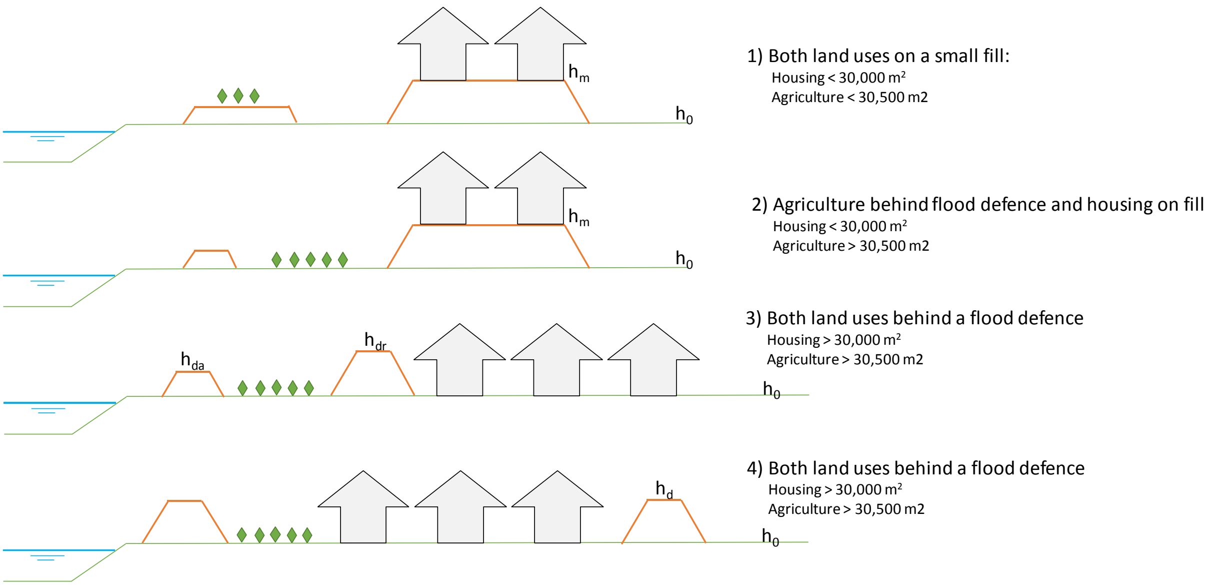

Figure 49: Combination of multiple land uses and risk reduction strategies in a floodplain

A combination of a flood defence and a fill is only preferred if one land use is smaller than the corresponding transitional area and the other larger. For example, a residential area smaller than its transitional area can be placed on a fill, while the surrounding agricultural land (which is smaller than its transitional area) is protected by flood defences. This is illustrated in the second sketch in Figure 49. From an economic point of view, a combination is also preferred if the agriculture area is smaller than its transitional size, while the residential area is larger than its transitional size. In this case it might be more practical to have both land uses be protected by a flood defence, if there is sufficient room for the agricultural land to be placed behind the flood defence (see fourth sketch in Figure 49). A single layer of protection is preferred for all other combinations.

We also find that optimal elevation levels for flood defences are lower for smaller land use values. As shown in the third sketch in Figure 49, the optimal elevation level of the agricultural area (h_{da}) is smaller than the optimal elevation level for the residential area (h_{dr}). For this case, it may be more economical to construct one flood defence to protect both the agricultural and residential area (as shown in the 4th sketch). As further discussed in chapter 4, additional planning considerations (e.g., water management and infrastructure) may also greatly affect the choice for the flood risk reduction strategy and its layout and implementation.

4.3.3 Developing behind an existing levee

An example case study is considered where an area requiring flood risk reduction is situated behind an existing flood defence. The question here is whether it is wise to raise the existing defence or invest in land fills. This is illustrated in the following figure:

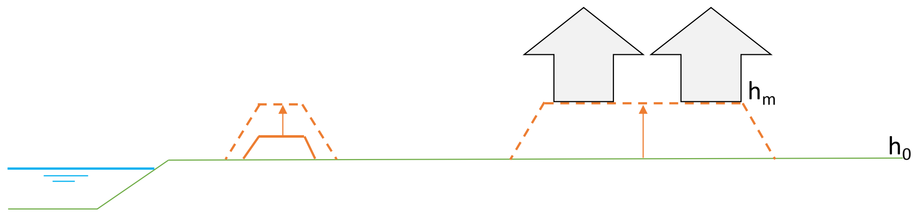

Figure 50: Developing behind an existing flood defence: reinforcing the flood defence or raising the floodplain on a fill?

The marginal cost for raising the entire residential area are estimated at 25 €/m/m^2. The marginal cost for reinforcing flood defences are estimated at 50% of the costs of constructing new flood defences (Table 17): 500 €/m/m. We consider an existing flood defence with an elevation level below the optimal elevation level (see Figure 51). For this case, higher optimal elevation levels are found compared to the base case scenario (Figure 44), as a result of the reduction of the marginal costs.

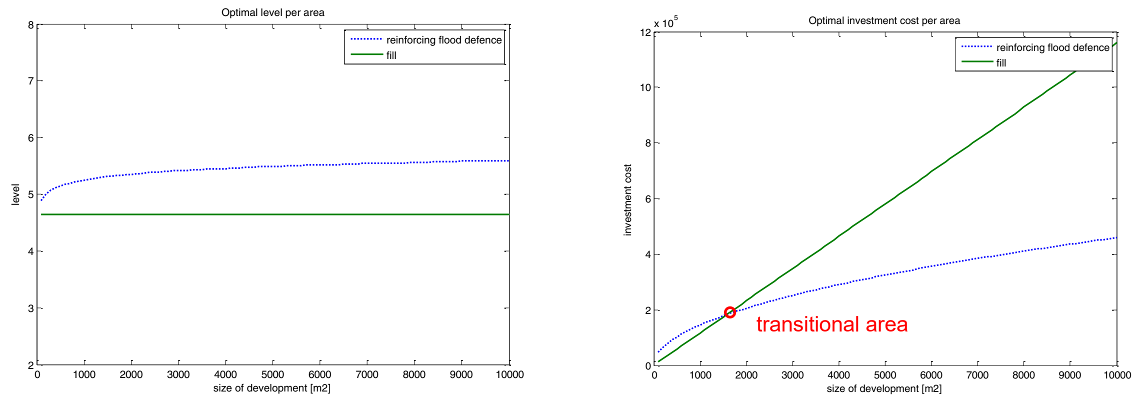

Figure 51: Optimal level (left) and total costs (right) for raising a fill or reinforcing the existing flood defences, given increasing size of the residential area. The existing flood defence level (3 meters) is below the optimal level (5 meters).

In addition, we find that raising a fill behind the existing flood defence is only economical for rather small areas (in the example we find a transitional area of about 1,500 m^2). Reinforcing the existing flood defences is preferred for larger areas. This example illustrates that the presence of flood defences results in a stronger preference for flood defences over fills.

4.3.4 Design for a minimum safety level

The preceding examples assumed that there is no minimum safety level and the optimization process continues until optimal levels are found. However, there are practical situations in which fixed safety levels are required for interventions for flood risk reduction, and no complete optimization is required. An example is shown based on flood management in the United States, where safety levels are based on an annual flood probability of 1/100 per year.

In those cases, optimization of the elevation level can become less relevant. However, it can still be interesting to consider whether consequence mitigation can be an attractive alternative to reach the same amount of risk reduction. The following graphs compare the total cost of raising fills and constructing flood defences, specifically for a flood probability of 1/100 year (corresponding with an elevation level of 4.3 meter in the base case). While a system of flood defences is more economical when the total costs are optimized, the figure shows that the costs of raising fills are lower than the costs of flood defences for the 1/100 year level (irrespective of the size of the area to be developed).

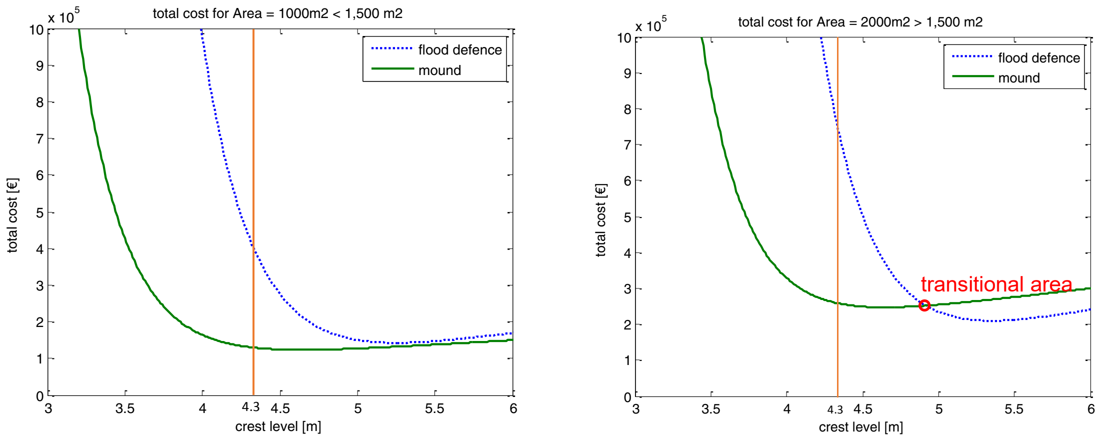

Figure 52: Total costs for a given area smaller than the optimal area (left) and larger than the optimal area (right), for increasing level of protection. Specifically shown is the crest level associated with an annual probability of flooding of 1/100.

4.4 Discussion

4.4.1 Results and practical implications

The concepts and conclusions drawn based on these results are generally applicable, however, the exact numbers should only be considered as an illustration. The results for the base case are based on typical values for western countries such as the Netherlands, with relatively high investment costs and damages, resuling in a strong preference for flood defences. The model framework can also be applied to other cases and applications, such as new developments in developing countries. These will be typically characterized by relatively low investment costs and damage densities, leading to a stronger preference for fills, because the transitional area where flood defences are preferred over fills is much larger.

Country – specific values for costs (e.g., Jonkman et al. 2013) and damage values can be used to come to more realistic local applications and the local “demand for safety” and need for coastal adaptation (Hinkel & Nicholls 2010). The model concept can serve as a basis, but more realistic inputs for elevation, damage density and investment costs could be incorporated – often necessitating numerical elaboration. Also, in further localized studies a broader set of interventions could be incorporated, including protective measures such as storm surge barriers, nourishments, reefs and different forms of damage reduction.

While this study focused on an economic-engineering consideration of risk, other drivers may determine which strategy is preferred. For example, an advantage of flood defences is that these are adaptable to changing boundary conditions, for example due to sea level rise or subsidence. It is easier to raise existing flood defences than to raise an entire fill. In many parts of the world, subsidence is a significant risk enhancer. An important aim could be to prevent or minimize future settlements, which are larger when fills are constructed compared to flood defences (fills have a larger footprint and therefore higher pressures on the subsoil). Raising fills on weak subsoils can become very costly and time consuming, if soil replacement is needed to prevent large settlements.

A system of flood defences surrounding a polder requires a water storage and drainage system, to drain excess water (due to rainfall or seepage) out of the polder. The operation and maintenance cost of such a system can become costly in areas with significant rainfall, which may drive decision makers to choose for fills. Another driver that may influence decision making are the higher potential flood damages behind flood defences once the elevation level of the flood defence is exceeded. Decision makers may want to prevent such hazards partly or entirely. Such risk aversion among decision makers can be included in optimization models, as was shown by Slijkhuis et al (2001).

Time constraints may drive decision makers to choose for costlier strategies, in order to finish a project earlier, achieve protection and / or start generating revenue. An example is the Tanjung Priok port terminal in Jakarta, where a deck on piles was built to avoid large settlements of a large fill (IPC port developer 2012). A large fill or polder may have been more economical (Lendering et al. 2015), but would probably have resulted in significantly longer construction times. Operational considerations may also play a role. For example, for port expansion projects, fills may be preferred over polders, because a level difference between the outer flood defence (the quay wall) and the terminal yard gives longer turnover times and higher operational costs (Lendering et al. 2015).

One of the most significant drivers of decision making remains the budget available for risk reduction. In the Netherlands, almost the entire county lives in so-called dike rings, and everyone pays a “water tax”, which is used to construct and reinforce existing flood defences. In other countries, such as the UK and US, only projects with high benefit cost ratios are funded. The practical examples all assumed that there is sufficient budget to raise entire areas to optimal elevation levels, irrespective of their cost. However, budgets are often constrained, which may drive decision makers to choose other than optimal strategies that are within the boundaries of the budget.

A final driver of the choice of a strategy for flood risk reduction concerns the governmental context. Flood protection of larger areas often relies on collective efforts, often necessitating the formation of water authorities and taxation. Consequence reduction by land fills or raising of structures is more easily achieved at the local level, up to the individual household.

4.4.2 Methodological considerations

The sensitivity of the results to the *location* and *scale* parameters of the exponential water level distribution was not analyzed. Instead, all examples were based on the distribution parameters included in Table 17, which are representative for a relatively mild exponential distribution as found in the North Sea. Steeper exponential distributions, with larger *scale* parameters (e.g., areas subject to hurricane storm surges), can be found in other areas around the world (see Xian et al. 2018 for a comparison between New York and Shanghai). Larger scale parameters result in higher optimal flood probabilities and higher risk (and vice versa for smaller *scale* parameters). Due to the assumed values of other parameters (e.g., marginal costs and land use values), significant changes in the exponential distribution do not affect the general conclusions drawn. Nevertheless, for local applications, more research in the statistical parameters is recommended to validate the results, possibly requiring a numerical analysis.

The mathematical model derived in this chapter assumes that the marginal cost of flood defences depends on the length and height of the flood defence. More detailed analysis can also take other cost drivers in to account, such as the total volume of soil, the outer slope protection and the type of flood defence. Furthermore, we assumed a circular polder, while in practice different shapes are found and built. While different shapes may change the analytical derivations found, we expect the impact on the principal results to be small. Another simplification was to assume that the indirect damages associated with flooding were part of the direct damages, which is in line with the approach proposed by Hallegatte (2013). More detailed methods are based on the potential income of the area for a specific period of downtime due to flooding or input output modelling (Steenge & Bockarjova 2007).

Finally, loss of life was not considered. In literature, the value of human lives has been included in comparable cost benefit analyses, for example by valuating human lives by the nett national product (NNP) per inhabitant (Vrijling et al. 1998; Jonkman et al. 2003). Another method to include loss of life in the design of flood risk reduction strategies is to require minimal safety levels to satisfy maximum individual risk standards (Jonkman et al. 2011).

4.5 Concluding remarks

This chapter expands existing methods for optimizing the total cost of flood defences and fills, by deriving largely analytical solutions for optimal elevation levels. Variations in the size of the area to be developed, its land use and corresponding value are included to model the total costs more accurately. The derived equations allow for optimization of a single strategy (i.e., a flood defence or a fill), and combination of interventions (i.e., a fill behind an existing flood defence). Using these equations, several practical examples of decision problems in flood risk management have been elaborated and implications for developing and developed countries have been discussed.

Within the context of this economic model, we conclude that a system of flood defences is more economical than a land fill for larger areas (above an identified transition level). Fills are preferred for specific combinations of areas and land uses, or when low flood safety levels are required. The ratio between the marginal cost of fills and flood defences largely determines the size of the area for which flood defences become more economical. An increase of the marginal cost of fills leads to a reduction of its application range from an economic point of view (and vice versa).

The practical examples in this chapter demonstrate that investing in a single protective layer (fills *or* flood defences) is generally more economical than combining multiple protective layers (fills *behind* flood defences). Nevertheless, combinations of inteventions can be attractive for specific cases. An example of such an optimal multi-layered strategy is found, when the value protected by the flood defence is low (agriculture) and the value protected by the fill is high (human lives/housing/ industry) and if the high value development is relatively small in size. In this case it makes sense to develop on raised land fills behind flood defences. The proposed approach gives insights in tipping points between optimal strategies and the sensitivity for the main problem characteristics.

The derived methods have focused on land fills but can also be applied to similar strategies such as floodproofing of structures or raising houses. As such, it is relevant for different areas subject to flood risks around the world (e.g., the Vietnam deltas or Japan coasts). Besides economic optimization of strategies, local requirements (limited rainfall flooding or settlements) or other drivers (e.g., time or budget constraints) may influence decision makers in deciding between different strategies.

The model concept could serve as a basis for local applications in decision support models, which may also contain aspects such as social, ecological impacts of interventions and governance. Also, in further localized studies a broader set of interventions could be incorporated, including protective measures such as storm surge barriers, nourishments, reefs and different forms of damage reduction. Such incorporations, combined with local inputs for elevation, damage density and investment costs, often necessitate numerical elaboration. Ultimately, this work serves as a basis to support decision makers in finding optimal strategies to manage and reduce flood risk.

5

Assessing the performance of flood adaptation innovations

The application of risk-based approaches for the design of flood infrastructure has become increasingly common in flood risk management. This approach, based on risk reduction and reliability, is used to assess the performance of conventional interventions (e.g., flood defences and dams) and to support decisions regarding their implementation. However, for flood adaptation innovations, performance has often not been quantified by means of these metrics and, therefore, end-users are hesitant to implement them in existing flood risk reduction systems. Here, flood adaptation innovations are defined as solutions that have not been assessed in terms of risk reduction and/or reliability, or solutions that have not yet been applied in practice.

This chapter presents a framework, based on four performance indicators, to ensure the required insights in risk and reliability are provided. The four indicators: effectiveness, durability, reliability and costs, allow end-users to evaluate, select, and implement flood adaptation innovations, and provide innovators with insight into the performance of the technology and the criteria and information necessary for successful market uptake of their innovation.

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5.1 Introduction

With recent climate observations suggesting that the frequency and intensity of flood events are increasing and growing urbanization in flood-prone areas , human exposure to floods (i.e., potential for loss of life) and flood damages continue to rise (Hallegatte et al. 2013). This trend of increasing flood risk is expected to continue during the 21st century (IPCC 2014).

To mitigate evolving flood risks, existing flood protection systems will need to be adapted and new systems designed and implemented. In addition to conventional forms of structural interventions for flood protection (e.g., flood defences and dams), innovative solutions offer the potential to reduce annual flood losses by decreasing flood risks. In other cases, innovative solutions may be critical for reducing risk in the short term while existing flood risk reduction systems are adapted or reinforced, or more comprehensive systems are built.

In flood management, the performance of structural interventions is commonly assessed based on risk reduction and reliability, and many methods and tools have already been developed and applied to do so. Examples include guidelines for the design and evaluation of levees (Ciria 2014), dams (FEMA 2004) and storm surge barriers (PIANC 2006; Mooyaart et al. 2014). In countries like the Netherlands, the United Kingdom and the United States, flood management policy is based on risk reduction, which often rely on set requirements for reliability (Schweckendiek 2015). Here, probabilistic risk-based approaches are developed and applied to establish safety levels and assess flood risk.

In this chapter, flood adaptation innovations are defined as solutions that have not been assessed in terms of risk reduction or solutions that have not yet been applied in practice. Examples include temporary flood barriers, green infrastructure and early flood warning systems. Due to limited experience with their operational performance, end-users are often hesitant to implement these innovations as key components in flood risk reduction systems instead falling back on more conventional interventions like sand bags and soil berms even though they have widely recognized limitations (Wibowo & Ward 2016; Lendering et al. 2016; de Leeuw et al. 2012). In addition, risk-based approaches often require information about the performance of solutions that is not typically provided by innovators. As a result, there is a knowledge gap between the information that end-users require when evaluating whether to implement an innovation and product-testing performed by innovators, hampering the widespread uptake of flood adaptation innovations.

Thus, the question of how to systematically analyze the performance of flood adaptation innovations within the risk-based framework has become increasingly important for their uptake. This chapter presents a framework for evaluating the technical performance of flood adaptation innovations based on their ability to reduce flood risk. By doing so, we aim to provide practical guidance to enable end-users to evaluate, select, and implement flood adaptation innovations. The framework also provides innovators with insight into the minimum criteria that should be provided to an end-user to facilitate market uptake. The framework was developed as part of the BRIGAID (BRIdging the GAp in Innovations for Disasters) Project, funded by the European Union through the Horizon2020 Programme. BRIGAID's aim is to develop a framework for evaluating the socio-technical performance of innovations for climate adaptation (Sebastian et al. 2017) because, specifically in Europe, there is no unified strategy for evaluating the performance of these innovations (European Commission 2015).

The chapter is organized as follows: Section 5.2 describes the basic principles of the risk-based approach and Section 5.3 describes typical flood adaptation innovations and how they are integrated into flood risk management. Section 5.4 presents the framework for assessing the performance of flood adaptation innovations based on four performance indicators: effectiveness, durability, reliability and cost; and Section 5.5 provides a case study with three examples of innovations for which the framework is applied. Section 5.6 discusses the effectiveness of this approach and limitations for implementation of the framework, while Section 5.7 presents the findings and directions for future research.

5.2 Basic principles of the risk-based approach

Traditionally, flood risk management is based on a safety-oriented approach in which structural measures (e.g., levees and storm surge barriers) are built to protect to the height of a design flood (Schumann 2017). The safety-oriented approach relies primarily on the quantification of the hazard for a given return period (i.e., the design flood) and assumes complete flood control. Because the probability of events larger than the design flood is small, the risk behind a structure is (generally) ignored (Figure 53) (Ludy & Kondolf 2012). In this case, it would imply that events with probabilities of 1/500 (corresponding to the design level of the defence) and smaller are ignored. The safety-oriented approach is currently used as the basis for decisions regarding flood mitigation in the United States, where flood insurance is only mandatory for federally-mortgaged structures in the 100-year floodplain and areas located behind levees are removed from the floodplain maps and considered to be safe. Currently, there are calls to move towards a more risk-based approach in the United States (Jonkman & Kok 2008b; NRC 2013a; NRC 2014).

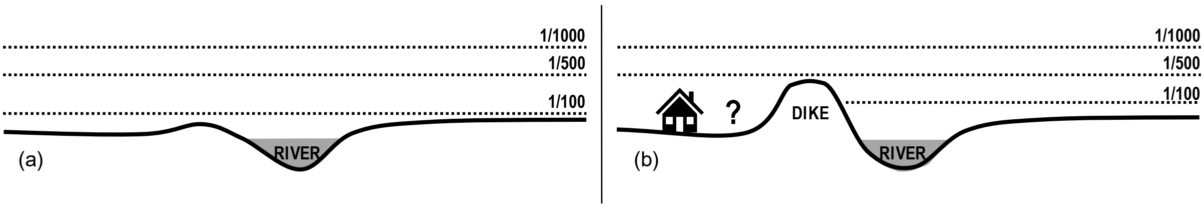

Figure 53: Annual probability of flooding in a river system without (a) and with (b) flood defences.

Within a risk-based approach, interventions in flood risk reduction systems are often compared based on their potential to reduce annual flood risk. While the definition of risk varies across different disciplines (Klijn et al. 2015), herein annual risk is defined as the product of the annual probability of a hazard and its potential adverse consequences, where consequences are a function of the exposure of, for example, people, buildings, and infrastructure to the hazard and their vulnerability (i.e., engineering, economic, social, environmental vulnerability) (Cardona et al. 2012; Traver 2014; Klijn et al. 2015). In theory, to assess the flood risks associated with a risk reduction system, all scenarios that may lead to flooding (e.g., coastal, pluvial and fluvial) are considered. Following this definition, annual flood risk is found by the summation of the risks associated with each scenario (Eq. 5.1).

$$\textit{Flood Risk} = \sum_{i=1}^{n} Annual\ probability\ (i)\ \cdot\ Consequences\ (i) \qquad (5.1)$$

Thus, the risk-based approach allows for cost benefit analyses of interventions, where benefits are expressed as a reduction of annual risk. As an example, the cost of building or raising flood defences can be compared and optimized against the damages avoided (i.e., annual benefits). This method was used by van Dantzig for the derivation of safety standards of flood defences in the Netherlands (van Dantzig 1956). In this way, van Dantzig showed how the risk-based approach and the safety oriented come together: the risk-based approach was used to derive safety standards, where the probability of overtopping was used as a proxy for the probability of flooding.

In the Netherlands, advanced probabilistic methods have been developed that not only take the probability of overtopping into account, but also other failure mechanisms of the flood defence (e.g., piping and instability) (Rijkswaterstaat 2016). Using these methods, updated safety standards for flood defences have been derived based on economic damages and loss of life (Rijkswaterstaat 2015; Jonkman et al. 2005; Slijkhuis et al. 2001; Jonkman 2007; Vrijling et al. 1998a; Kolen 2013). The new methods constitute a significant advance in the field of flood risk management (Vrijling 2001) and provide opportunities to include the effectiveness of previously neglected solutions in the reliability and risk assessment of flood defences as shown by Lendering et al. (Lendering et al. 2016).

Outside of the Netherlands, other countries have also made progress in developing methods and tools for assessing risks and reliability of flood defence systems, for example in the UK (Hall et al. 2003), USA (USACE 2009) and in the Shanghai region in China (Jiabi et al. 2013). Overall, it can be observed that the insights from risk and reliability analyses are now at a stage that they can be more directly applied in policy making (e.g., safety standards) and the design and management of flood defences (Schweckendiek 2015).

5.3 Flood adaptation innovations

Risk is constantly evolving (dependent on increasing hazard loads, urban development patterns and economic changes) requiring fast adaptation to prevent risks increasing beyond acceptable levels. Intense use of protected floodplain areas previously perceived to be completely safe can cause risk levels to grow beyond what was previously calculated, while the rising costs of floods globally have drawn attention to the potential for damages even in protected areas (Tarlock 2012; Costa 1978).

A flood risk reduction system aims to reduce flood risks by decreasing the probability of flooding and its consequences. A wide range of solutions are available to reduce flood risks. In practice, solutions are often categorized as part of one of three layers of risk reduction: (1) protection, (2) prevention, and (3) preparedness (Figure 54) (Kolen & Kok 2011; Kolen et al. 2012). In the context of risk as defined in Eq. 5.1, protective measures reduce the probability of flooding through structural measures (e.g., flood defences), whereas preventive and preparedness measures address the consequences of flooding through, for example, spatial planning and evacuation, emergency response, and recovery, respectively (see Table 1).

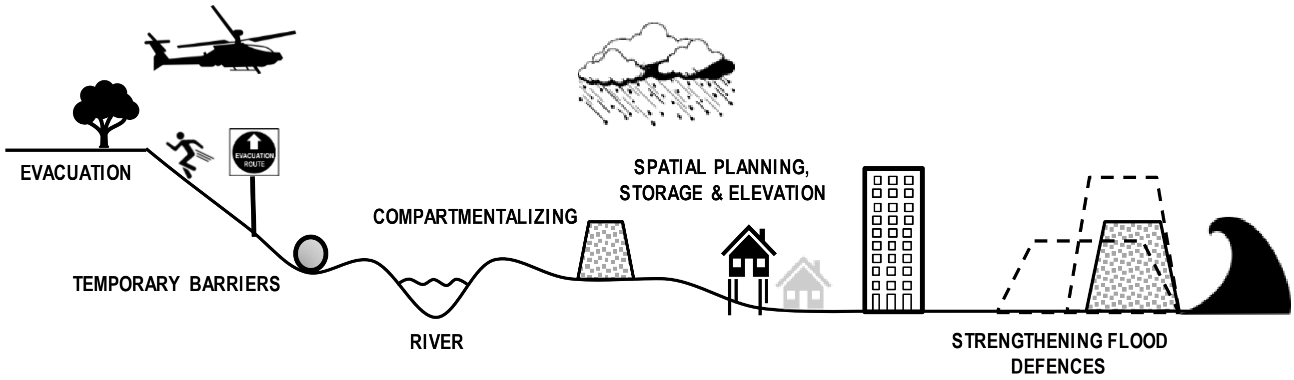

Figure 54: Integrated flood risk management and multi-layer safety: (1) prevention, (2) protection, and (3) preparedness.

Layer	Examples of Measures
Protection	dams; levees; floodwalls; dikes; seawalls; flood gates; floodways and spillways; channel modifications; storm water management; on-site retention; detention; breakwaters; bulkheads; groins; revetments; nourishments;
Prevention	spatial planning; safe land-use practices (e.g., setbacks); construction standards and building codes (e.g., vertical elevation); flood proofing; acquisition and relocation; coastal zone management; green roofs
Preparedness	forecasting; early warning; evacuation; emergency measures; temporary flood barriers; floodplain mapping; flood insurance; disaster relief; subsidies; public awareness and education

Table 19: Examples of solutions for reducing flood risk by layer.

The proportional investment in each of these layers varies between countries. For example, presently, the U.S. invests primarily in preparedness (e.g., flood insurance and evacuations), whereas the Netherlands is focused on protection (Bubeck et al. 2013). In the Netherlands, the up-front investment required for protection is much higher than prevention or preparedness, but the structural measures for protection are often calculated to be more cost-effective over the long term (Lendering et al. 2016).

Because flood risk management considers the risk reduction potential of all interventions in the system, interventions aimed at reducing flood risk behind protective structures have a marked potential for reducing flood risk at a system level. However, the implementation of innovative solutions within these layers is limited, due to the absence of and tools to evaluate the risk and reliability associated with these innovations. While advanced probabilistic methods to assess flood risk were developed and have been applied to interventions within the protective layer (and are thus straight-forward for these applications), they have not been widely applied (or tested) for interventions within the preventive and/or preparedness layers (e.g., for temporary defences (Lendering et al. 2016; Wibowo & Ward 2016)). Moreover, while end-users generally acknowledge the advantages of the advanced probabilistic methods, they remain computationally expensive (Dupuits et al. 2016) and specific applications require new extensions or adjustments of the current methods (Lendering et al. 2016).

As other countries also begin to move towards utilizing risk-based approaches to mitigate the economic impacts of natural hazards, there is a need for insight and research into the application of the risk-based approach to assess the performance of flood adaptation innovations. Thus, to demonstrate the application of the risk-based approach, we focus primarily on innovative solutions which are designed to be integrated in the preventive and/or preparedness layer of a flood risk reduction system. Some examples include small-scale green infrastructure (e.g., pocket parks, green roofs, and smart streets), temporary or mobile flood defences, and local flood warning or flood forecasting systems.

5.4 Framework for assessing performance of flood adaptation innovations

The move towards utilizing risk-based approaches to design integrated flood risk management systems requires performance-based planning of flood mitigation measures. Innovators aiming to market flood adaptation innovations are therefore required to provide the information necessary for end-users to evaluate their performance in terms of the risk reduction potential relative to existing risk reduction systems. End-users require such information before deciding whether to implement an intervention in the risk reduction system.

The framework demands "risk-informed decision-making," which must be based on aspects such as costs and benefits over the lifetime of the innovation, where benefits are expressed as damages avoided (i.e., annual risk reduction). In the cost benefit analysis, costs are balanced by obtained risk reduction from an economic point of view. Costs are determined by an innovation's investment costs (I) and its annual operation and maintenance cost. Cost-effectiveness is evaluated based on a comparison of an innovation's ability to reduce flood risk (i.e. ΔR = flood risk reduction) against its cost (C) discounted over the innovations lifetime. Here, risk reduction is expressed as the present value of avoided damages (ΔEAD) discounted over the lifetime (T) of the innovation, while costs are determined by an innovations investment cost (It=o) and the present value of the operation and maintenance cost (O&M) discounted over the lifetime (T) taking a discount factor (r) into account.

Cost < Risk reduction = C < ∆R *(5.2)*

where C = It=0 + $\sum_{t=1}^{T}\frac{O\&M}{(1+r)^t}$ *and ∆R = ∆ EAD =* $\sum_{t=1}^{T}\frac{\Delta\,(P_f\cdot D)}{(1+r)^t}$

Several challenges have to be addressed in order to allow for risk-informed decision making. First, insight is required into the risks associated within the existing system. Second, a framework is required that allows innovators to systematically analyze the performance of the innovation within the risk-based framework.

Finally, the performance of the entire risk reduction system is analyzed with the flood adaptation innovation in place. To do so, testing within laboratory or operational environments is often performed to obtain data and information about the performance of the innovation, as experience with the practical performance of the innovation during a real hazard is often lacking. A framework for addressing these challenges is proposed in the following sections.

5.4.1 General Approach to assessing flood risk

The following section describes how flood risks are estimated based on the probability and consequences of flooding of an exposed area, more detailed guidelines can be found in CIRIA (2014), Rijkswaterstaat (2016), and Schanze (2006). While there are many different mathematical tools that can be applied during the process, the general framework shown in Figure 55 applies to all types of flooding (i.e., coastal, pluvial and fluvial) and measures.

An assessment of flood risk starts with a description of the risk reduction system (if any) and its boundaries. The considered system can have different scales ranging from large river deltas and coastal areas to smaller catchments and watersheds, or local sites. For the entire system, all scenarios that may lead to flooding are analyzed and described: extreme rainfall, rising water levels, failure of physical components of the system and/or organizational or process failures (Jonkman et al. 2015).

The system description is followed by a study of the probability of flooding, considering all scenarios that can lead to flooding and any interventions that have been applied to reduce the probability. For example, for fluvial and coastal flooding, the probability of flooding is determined by the probability of exceedance of a given water level in the river or sea. If flood defences were built, the probability of flood defence failure needs to be taken into account (e.g., due to overflowing or structural failure) (Jonkman et al. 2017). For pluvial flooding, the probability of flooding is calculated based on the probability of a given water level occurring driven by a rainfall event of a certain intensity, duration, and frequency. Similarly, any interventions (e.g., increasing drainage capacity) that increase the capacity of the system to handle pluvial flooding or reduce flood impacts need to be considered when calculating the probability of flooding.

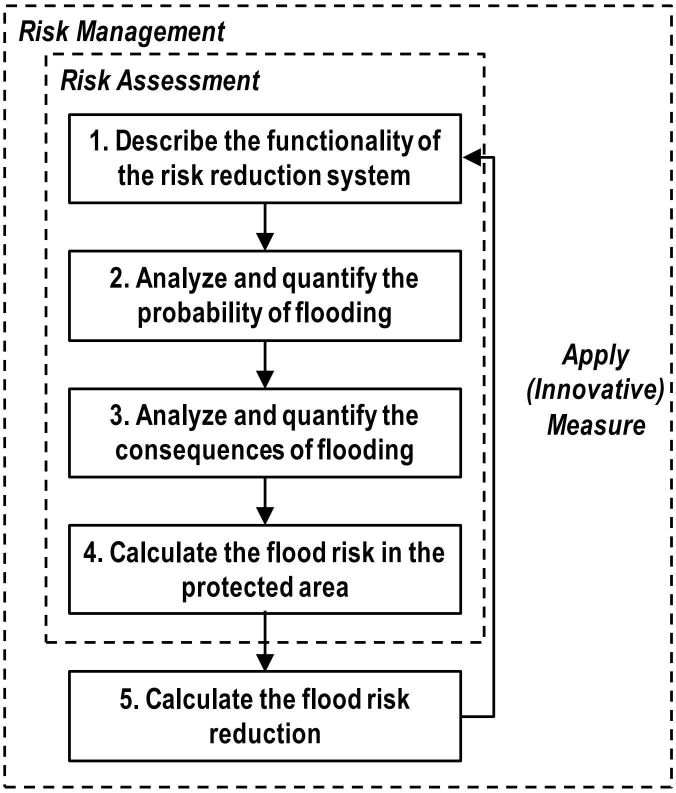

Figure 55: Assessment of flood risk in an existing system (Steps 1-4) and risk reduction of an (innovative) measure (Step 5).

The consequences of flooding are assessed by simulating inundation levels and quantifying the potential consequences in monetary terms, considering both direct (material) damages and indirect (economic) losses. The risk is then quantified by multiplying the probability of flooding of all scenarios with their potential consequences and summing the risk associated with every scenario. An evaluation of acceptable levels of risk often considers three criteria: risk to individuals, society and the economy (Vrijling et al. 1995; Vrijling et al. 1998b). According to Vrijling et al., decisions regarding acceptable levels of risk should be based on the most stringent of the three criteria (Vrijling et al. 1998a). Flood adaptation innovations are applied if risks are deemed too high. After application, the risk and reliability associated with the specific scenario are reassessed with the innovation in place. This cyclic process is followed until end-users find the risk to be reduced sufficiently. Part of this process may be to make changes to the considered innovation to increase its effectiveness. Such changes could consider the implementation or operation process, the technical design or operation and maintenance protocols. Ultimately, innovators will continue this cyclic process until the end-user conditions are met.

5.4.2 Performance Indicators

To provide the necessary information to support risk-informed decision making, four performance indicators (PI) are used: effectiveness, durability, reliability, and cost (Table 2). In developing these PIs, different frameworks for evaluating the performance of different types of innovations were reviewed, including temporary flood barriers (Margreth & Romang 2010; Lendering et al. 2016; Wibowo & Ward 2016) and early flood warning systems (Sättele et al. 2015; Sättele et al. 2016). While recognizing that tests and results for individual innovations may vary, the PIs are generally applicable and relevant for all flood adaptation innovations. Note that the here proposed methods serve as an example; other methods can be used (and could be more effective) when analyzing the performance of different types of innovations, so long as the required insights of each indicator are provided.

Indicators	Definition	Parameter
Effectiveness	A metric that describes the intended capacity of the innovation to reduce flood risk, either by reducing the probability (P_f) or consequences (D) of flooding in the exposed area.	ΔP_f or ΔD
Durability	A metric that encompasses the temporary- or permanent-nature of the innovation and its operational lifetime (T) and provides insight in its flexibility of use.	T
Reliability	A metric that describes the likelihood that an innovation fulfils its intended functionality during its intended lifetime ($P_{f;innovation}$).	$P_{f;innovation}$
Cost	A metric that describes the costs (C) associated with the purchase, installation and operation (and maintenance) of the innovation over its lifetime.	C

Table 20: Description of Performance Indicators used to analyze the effectiveness of flood adaptation innovations within the risk-based framework and their corresponding parameters in Eq. 5.2.

Effectiveness

Effectiveness is a metric used to evaluate the intended capacity of the innovation to reduce flood risk either by reducing the probability of flooding of the exposed area or by reducing the potential consequences of flooding (Eq. 5.1). For example, a temporary flood barrier provides protection for water levels up to its height, thereby increasing the design water level and reducing the flood probability. A green roof prevents large run-off flows by providing temporary storage capacity during heavy rainfall, which also reduces the flood probability. In comparison, an early flood warning system provides more lead time in anticipation of a flood to allow for more effective preparation (e.g., evacuation or flood fighting), which reduces the flood consequences.

The approach described here to quantify the effectiveness requires innovators and end-users to describe/ analyze how the innovation interacts with the existing flood risk reduction system and assess the resulting risk reduction in terms of a reduction of the probability (ΔPf) or consequences of flooding (ΔD). For example, considering a temporary flood barrier used to temporarily heighten levees during a river flood. The obtained reduction of the probability of failure can be assessed using fragility curves for failure of the considered levee, which illustrate the conditional failure probability on the loads exhibited on the innovation, as shown in Figure 56.

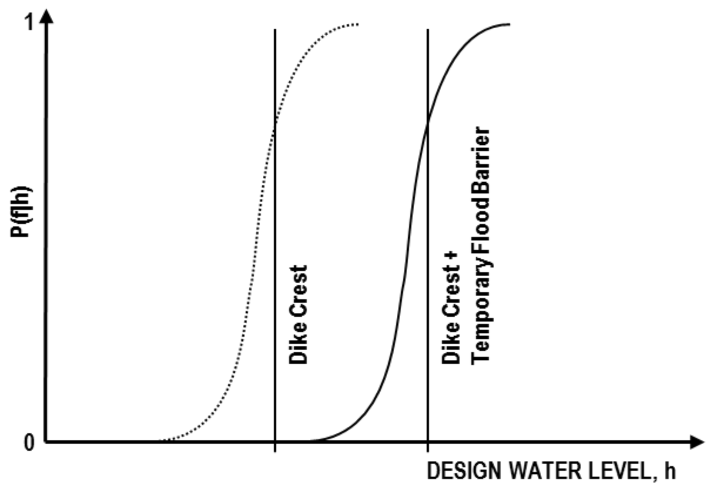

Figure 56: Using fragility curves to demonstrate the potential effectiveness of temporary flood barriers used to increase a dike crest

The described approach to determine effectiveness assumes successful implementation of the innovation, but foregoes the probability of failure of implementation of the innovation itself. Innovations may fail due to failure of installation, operation or technical failure. These aspects, as well as the innovation's durability, are taken into account within the durability and reliability indicators.

Durability

Durability is a metric that encompasses the lifetime of an innovation and describes the temporary- or permanent-nature of the operation of the innovation. It takes into consideration how durable the structural components of the innovation are and whether the innovation is designed for single or repetitive use. Innovations designed for repetitive use may be operated permanently (i.e., continuously) or temporarily (i.e., only during the flood hazard). Assessing the durability of the innovation requires estimating the (percentage of) components that require repair or replacement after each operation of the innovation (if designed for repetitive use).

Together, these aspects provide insight in the lifetime of the innovation — determined by either the lifetime of its structural components or the innovation's climate lifetime — and the long-term operation and maintenance requirements to meet that lifetime. Here, an innovation's climate lifetime is the time at which its performance (i.e. its intended capacity to reduce flood risk) is exceeded by climate change impacts. For example, the climate lifetime of a temporary flood barrier is exceeded when the barrier's height has been exceeded by increased water levels (e.g., due to sea level rise, Figure 57).

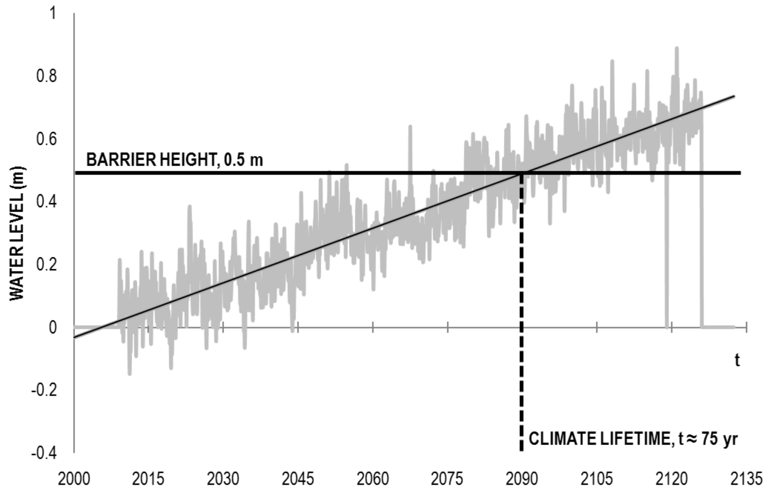

Figure 57: Climate lifetime (t_0 = 2015) of a 0.5 m barrier determined by water levels rising at an average rate of 6 mm/year.

The ability for repetitive use of an innovation provides a certain flexibility in the application of innovations. For example, innovations that are temporary (and deployable) in nature and can be removed after an event or used at multiple locations are much more flexible than conventional permanent measures. An additional benefit of this flexibility is that such innovations can be adaptable to different loading conditions (e.g., increased loads due to climate change) over their lifetime.

Reliability

Reliability is a metric that estimates the likelihood that an innovation fulfills its intended effectiveness during its intended lifetime. By definition, reliability is the probability of successful operation, which can also be expressed as the complement of the probability of failure during operation (i.e., reliability = 1 – probability of failure during operation). Here, failure is described as the inability of the innovation to fulfill its intended function. For example, the reliability of a temporary flood barrier is evaluated by determining the probability that the barrier fails due to failure of mobilization, placement, or failure to retain water levels up to its design height. Similarly, the reliability of an early flood warning system is evaluated by determining the probability that the system (or its components) is unavailable or that the system fails to predict flooding (Sättele et al. 2015).

To analyze failure modes, all (known) undesired events that may cause failure of the innovation should be identified. Distinction is made between two main failure modes: implementation failure and technical failure (Figure 58). Implementation failure only applies to innovations that are operated temporarily and is defined as failure to implement the innovation before operation (e.g., due to logistical failure (de Leeuw et al. 2012) or operator error (Corn & Inkabi 2013)), whereas technical failure is defined as failure of the innovation to fulfill its intended function during operation (e.g., due to structural component failures). Typical methods used to analyze and understand how implementation and technical failure of innovations may interact include failure mode and effect analyses (FMEA) or failure mode effect and criticality analyses (FMECA) (Ciria 2014).

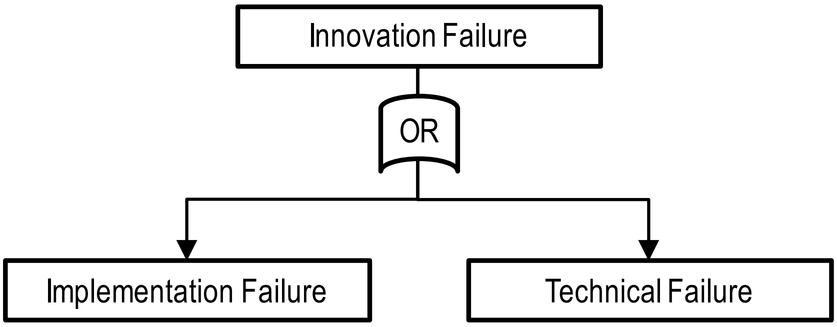

Figure 58: Example fault tree including implementation and technical failure

Probabilistic methods are used to quantify an innovation's reliability. The failure probability of systems that rely on human actions (i.e., operators) is often dominated by the probability of operator errors, which is estimated using Human Reliability Analyses (Bea 1998; Rasmussen 1983). These analyses typically seek only order of magnitudes of probabilities of failure. Lendering et al. (2016) developed a method for assessing the probability of human errors during implementation of emergency measures for flood prevention, which can also be used for flood adaptation innovations. In addition, methods were developed to assess the probability of logistical failure, taking into account the available time for implementation. Finally, the probability of technical failure modes, such as component, hardware, software or structural failure, can be estimated for every technical failure mode using probabilistic methods such as Monte Carlo Simulations or First Order Reliability Methods (Jonkman et al. 2015). For warning and operation systems, software and organizational reliability become a part of the overall assessment (Bea 1998). For these analyses, innovators are required to describe and analyze their innovation and provide data that can be used to estimate probabilities of failure.

Cost

Costs are determined by the investment cost (It=0) and the operation and maintenance costs (O&M) over the innovation's lifetime (T). The investment costs depend on the costs of the material components and the initial installation costs of the innovation, while the operation and maintenance costs depend on the innovation's durability: whether the innovation is operated continuously or temporarily (and how often); whether the innovation require repairs after each use (and how much); and its intended technical or climate lifetime. Note that for an innovation designed for temporary use, the annual operation and maintenance cost are determined by the number of times the innovation is used per year multiplied by the associated cost. The following equation determines the present value of the cost of the innovation over its lifetime, considering a discount factor (r):

$$C = I_{t=0} + \sum_{t=1}^{T} \frac{O\&M}{1+r^{t}} \qquad (5.3)$$

5.4.3 Performance Assessment

The obtained risk reduction (ΔR) with the innovations in place is measured relative to the existing flood risk reduction system (including any measures that are already in place). It is measured as a function of the overall risk of the considered scenario without the innovation in place. Depending on how the innovation reduces risk (i.e., by reducing the probability or consequences of flooding), its effect is included in the assessment of probability or in the consequences of flooding of that specific scenario. For innovations that focus on reducing flood probabilities (i.e., prevention), the obtained risk reduction is calculated as follows:

$$\Delta R = \sum_{t=1}^{T} \frac{(Pf;old - Pf;new) \cdot D}{1+r^t} \tag{5.4}$$

where $P_{f;new}$ represents the new probability of flooding with the innovation in place and $P_{f;old}$ represents the probability of flooding without the innovation in place.

The probability of flooding with the innovation in place is calculated using the total law of probability, taking into account both the effectiveness and reliability of the innovation. The probability of flooding with the innovation in place considers two scenarios: successful operation of the innovation and failure of the innovation:

$$P_{f;new} = P_{f;innovation} \cdot P_{f;old} + (1 - P_{f;innovation}) \cdot (P_{f;old} - \Delta P_f) \tag{5.5}$$

For innovations that are designed to reduce the consequences of flooding, risk reduction is calculated as follows:

$$\Delta R = \sum_{t=1}^{T} \frac{Pf \cdot (Dold - Dnew)}{1+r^t} \tag{5.6}$$

where D_{new} represents the potential damages of flooding with the innovation in place and D_{old} represents the potential damages of flooding without the innovation in place.

The potential damage of flooding with the innovation in place is estimated considering both successful operation of the innovation as well as the likelihood of innovation failure:

$$D_{new} = P_{f;innovation} \cdot D_{old} + (1 - P_{f;innovation}) \cdot (D_{old} - \Delta D) \tag{5.7}$$

By comparing the resulting risk reduction to the costs associated with the innovation, end-users are able to evaluate the costs and benefits of the innovation over the intended lifetime (T). Innovations are cost-effective when their cost is lower than the present value of the expected damages over the considered lifetime (Eq. 5.2).

5.5 Application in practical situations

To demonstrate the application of the proposed framework in practical situations, the framework was applied within a given, fictional, case study. We consider a large hospital complex built in an area of about 0.24 km2 which has subsided approximately 2 meters below the surrounding area. The hospital facilities cover about 75% of the total area and the total value of the hospital complex is estimated to be €1 billion. The area is subject to tropical rain showers which can result in flash flooding due to insufficient drainage capacity in the surrounding area. Statistical analysis of rainfall intensities resulted in the intensity-duration-frequency (IDF) curves shown in Figure 59.

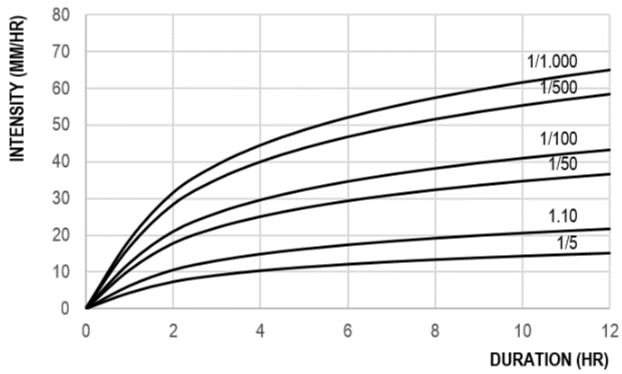

Figure 59: Intensity-duration-frequency curves for fictional case study of a large hospital complex.

Using the intensity-duration-frequency curves for rainfall, return period water levels were estimated for the area, as shown in Figure 60. We assume that the system is closed, and that negligible infiltration is occurring. Figure 60 shows estimated material damages dependent on the depth of flooding and expressed as a fraction of the total value. The annual risk of flooding is found by integrating the damages associated with different return periods and summing (Eq. 5.1), resulting in a value of €22 million.

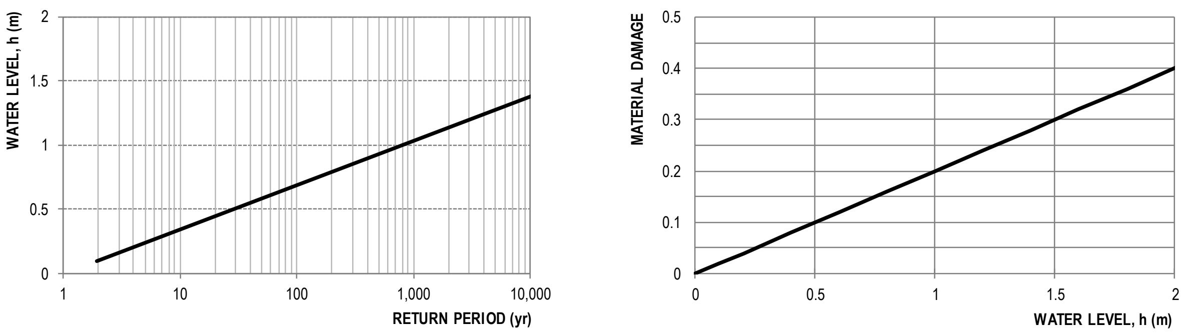

Figure 60: Return period of water levels (left) and flood damage estimates (right) expressed as a fraction of the total value of the hospital complex.

To reduce flood risk to the hospital complex, several flood adaptation innovations are considered consecutively: a flood warning and operation system to increase lead time and management of flood risks, green roofs to delay runoff and reduce pluvial flooding, and temporary flood barriers to protect hospital entrances.

5.5.1 Flood Warning and Operation System (FWS)

Currently, no flood warning systems are implemented in the area. Pluvial flooding may occur unexpectedly, leaving little time for any mitigative measures to be put in place. The hospital is considering implementing an early flood warning system (FWS) that provides a lead time of 4 hours for pluvial floods. An example of such a solution was implemented at the Texas Medical Center (Fang et al. 2014). The lead time provided by the FWS allows the hospital to close existing submarine doors to the parking garage under the hospital and prevent critical facilities from flooding. During previous flood events, little to no warning and lack of protocol resulted in the submarine doors being left open, rendering them ineffective for reducing flood losses. A description of obtained results for each performance indicator is included in Table 3.

Indicator	Description	Variable	Value
Effectiveness	The effectiveness of the FWS is defined by its ability to allow for mitigative action in anticipation of a pluvial flood: in this case closing the submarine doors to protect critical facilities. Total potential damage avoided (ΔD) to the hospital complex is €10 million.	ΔD	€10 million
Durability	The FWS is operated continuously and has a technical lifetime of 5 years, after which it should be replaced or upgraded using state-of-the-art data and models. Operation of the early flood warning system does not require significant maintenance during its estimated lifetime.	t	5 yrs
Reliability	The system is operated continuously and has a predictive capacity of 99%. This means that it fails to predict flooding during 1% of the time.	$P_{f;innovation}$	0.01
Costs	The investment cost of the system amount to € 500,000. The operation and maintenance cost during its lifetime are estimated at 10% of investment cost, which amounts to €50,000 per year. The present value of the total cost is €732,000.	C	€ 732,000

Table 21: Assessment of the flood warning system (FWS) in terms of each performance indicator.

The annual obtained risk reduction is calculated using Eq. 5.4 and 5.7 and amounts to € 220,000. The present value of avoided damages due to implementation of the early flood warning system amounts to €1 million considering a discount factor of 2.5% and a lifetime of 5 years. The innovation's cost, determined by the investment cost (€500,000) and annual operation and maintenance cost (€50,000), are €732,000. These are lower than the benefit (€1 million); thus, the innovation is cost-effective, with a benefit/cost ratio of approximately 1.4.

5.5.2 Green Roof

In the baseline scenario, the construction of the hospital has resulted in a reduction of pervious surfaces by upwards of 50%. Response to precipitation is nearly instantaneous, resulting in pluvial flooding. To reduce flood risk, the emergency manager is considering installing innovative green roofs on many of the hospital facilities to retain water temporarily during rainfall events, thereby reducing the total volume of runoff into the area. We assume that the green roof is constructed using peat soils and calculate the rate of infiltration based on Horton and the associated parameters provided in Maidment (1993). Considering that the hospital facilities cover almost 50% of the area, and green roofs can be placed on 67% of the hospital complex (Figure 61), the green roof is able to capture the 5- and 10-year precipitation events and portions of the larger events (Figure 62). This results in a substantial reduction in flood levels water levels at the hospital facility (Figure 62). A description of obtained results for each performance indicator is included in Table 4.

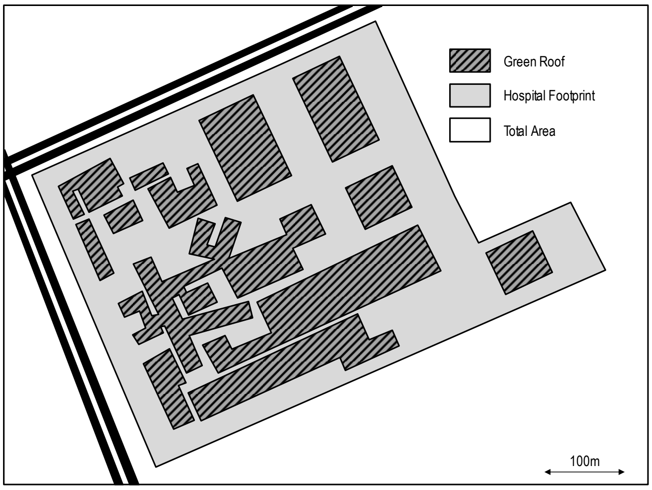

Figure 61: Area inside covered by the hospital, illustrating the area covered by green roofs

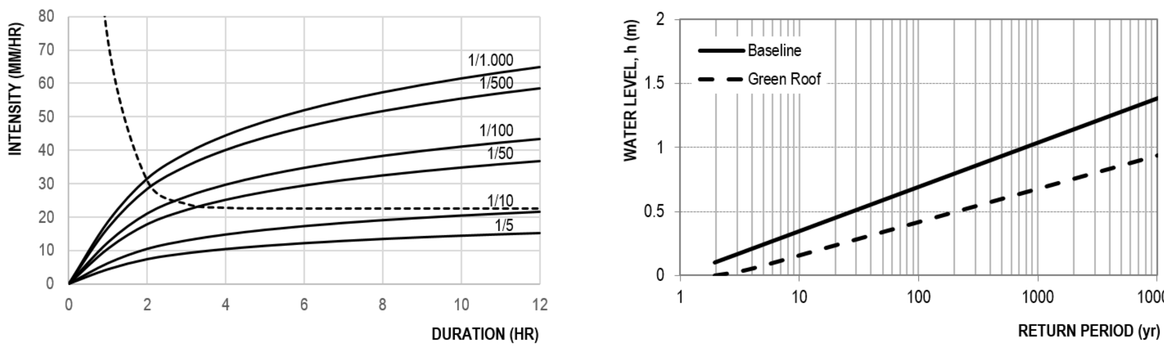

Figure 62: Intensity-duration-frequency curves for fictional case study illustrating the rate of infiltration achieved by the green roof (dotted line, left figure) and the resulting return period water level curves after installation of the green roof (right)

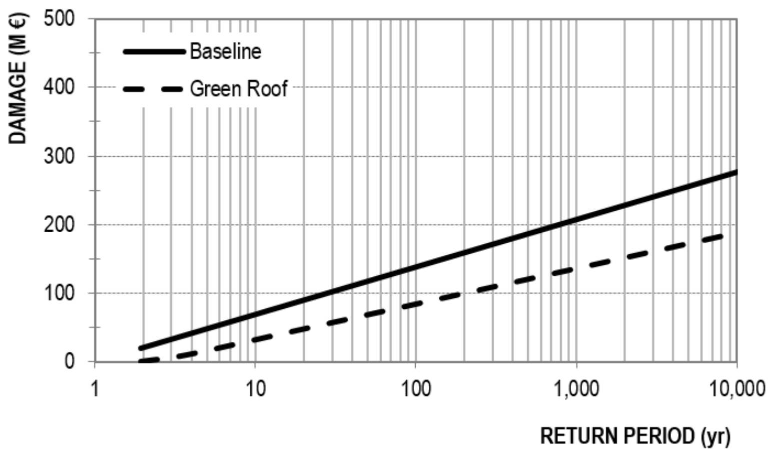

Figure 63: Resulting damage return period curves after installation of the green roof

Indicator	Description	Variable	Value
Effectiveness	The effectiveness of the green roof system is defined by its ability to reduce flood water levels by 0.2 to 0.5 meter for return periods ranging from 1/10 to 1/10.000 respectively. This translates to reduced damages (€ 25 to 50 mln) for these return periods as shown in Figure 63.	ΔD	€ 25 to 50 mln
Durability	The green roof system is operated continuously and has a technical lifetime of 10 years, after which it should be replaced or upgraded. Operation of the system requires annual maintenance of the release system.	t	10 years
Reliability	The system is operated continuously. Its probability of failure is determined by the likelihood of the green roof being fully saturated (i.e., not releasing stored water in time before succeeding rainfall events). Based on an analysis of the frequency of extreme rainfall events, the annual probability of failure is estimated to be 10%.	$P_{f;innovation}$	0.10
Costs	The investment cost of the system amount to €22.5 million, based on a unit cost of €250/m2 and a total area of 90,000 m2. The annual costs of operation and maintenance are estimated at 0.5% of the investment cost: €112,500 per year. Together, the present value of the cost amounts	C	€23.5 million

Table 22: Assessment of the early warning system in terms of each performance indicator.

The annual obtained risk reduction is calculated using Eq. 5.4 and 5.7 and amounts to € 10 million. The present value of avoided damages due to installation of the green roofs amounts to €89 million, considering a discount factor of 2.5% and a lifetime of 10 years. The innovations cost (€23.5 million) are lower than its benefits (€89 million), thus, the innovation is cost-effective with a benefit/cost ratio of 3.8. Its effectiveness can be further increased by increasing its storage capacity or reducing its investment and/or operational cost.

5.5.3 Temporary Flood Barrier

Temporary flood barriers can be applied to close the hospital entrance and prevent it from flooding. The conventional method for preventing flooding through the entrance is to use sand bags. However, the installation of sand bags is labor intensive, time consuming and sand bags are difficult to remove. In contrast, temporary flood barriers can be installed quickly prior to - and removed entirely after - an event. We consider water-filled tubes. The tubes provide protection up to their design height, typically 0.5 meter (see Figure 64), and obtain stability through the weight of water that flows inside the tube. An analysis of each performance indicator is included in Table 5.

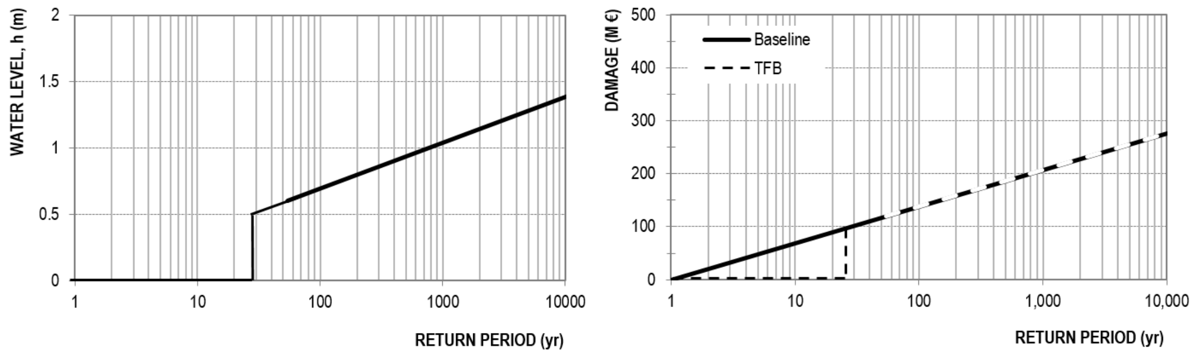

Figure 64: Return periods of flood levels with temporary flood barrier in place (left) and damage estimates for a situation with temporary flood barriers (dotted line) compared to the baseline situation. The return periods correspond with flood water levels.

To assess the probability of failure of the barrier, both implementation and technical failure are considered. Implementation failure may occur due to operator error or logistical failure (i.e., failure to transport the innovation to the required location), with operator error being the dominant failure mode. Technical failure can occur due to instability of the tube (e.g., due to sliding or turning over), ruptures of the canvas material, or seepage/leakage under the tube. Figure 65 illustrates a fault tree for the barrier. It is noted that this is a series system with OR gates, so all elements need to be sufficiently reliable to ensure adequate overall performance of the system.

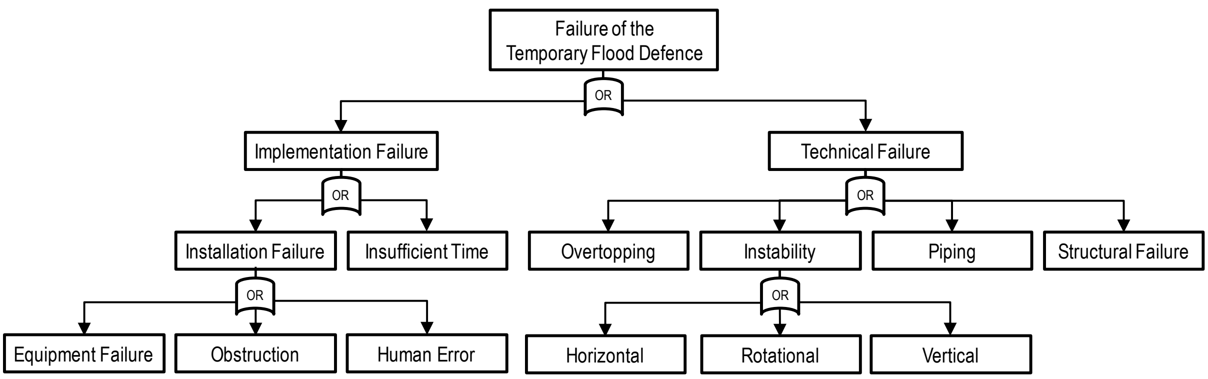

Figure 65: Example fault tree for a temporary flood barrier (TFB)

Indicator	Description	Variable	Value
Effectiveness	The effectiveness of the temporary flood barrier is defined by its ability to provide protection up to 0.5 meter, which corresponds to protection until the ~1/20 year event (Figure 63).	ΔP_f	1/20
Durability	The temporary flood barrier consists of plastic canvas material which has an expected technical lifetime of 20 years. It is estimated that after each use minor repairs (< 10%) to the tubes are required; such repairs could include patching a rip in the canvas material or replacing anchors.	T	20 years
Reliability	The operator error is estimated to be 1/50 per task according to the approach developed by Lendering et al. (2016) to assess the reliability of emergency measures for flood prevention, assuming the operator operates at a rule-based level (see Appendix C). Sliding failure (1/50 per use) will be governing considering the smooth surface of the entrance tiled floor. Assuming that the implementation and technical failures are independent, the probability of failure of both failure modes can be summed. The resulting probability of failure is 1/25 per use. Assuming the barriers are applied for every 1-year event, this results in an annual failure probability of 0.04 (or 1/25).	$P_{f;innovation}$	0.04
Costs	The investment cost of the system amounts to €10 million. The annual operation and maintenance cost amount to 10% of the investment cost. Over its lifetime, the total cost of the barrier amounts to €25.6 million.	C	€25.6 million

Table 23: Assessment of the water-filled tube barrier in terms of each performance indicator.

The annual obtained risk reduction is calculated using Eq. 5.4 and 5.7 and amounts to €18 million. The present value of avoided damages due to operation of the temporary flood barriers amounts to €310 million, considering a discount factor of 2.5% and a lifetime of 20 years. The innovation's cost (€25.6 million) is lower than its benefits (€310 million), with a benefit/cost ratio of 12.

5.6 Discussion

Of the three considered innovations in the fictional case study, temporary flood barriers are found to be the most cost effective suggesting that when trying to immediately reduce risks, these should be implemented first. These results should be considered in light of the considered case study and its characteristics. Areas subject to different flood hazards, such as, coastal or fluvial flooding, may give different results. The case study examples demonstrate the necessity of clusters or combinations of innovations because a single innovation is not always able to reduce flood risk. For example, a flood warning system on its own cannot reduce structural damage to the hospital, but, when used in combination with submarine doors or temporary flood barriers, a flood warning system has the potential to achieve a higher cost-benefit ratio than the other alternatives applied alone. Ultimately, the performance of flood adaptation innovations should not be evaluated in isolation, but always considering the existing risk reduction system.

In the case study we applied the innovations successively. For example, in considering the performance of the temporary flood barrier, we assumed that a flood warning system is in place and that its reliability is captured within the failure mode “insufficient time” (Figure 65). In reality, the successful operation of temporary flood barriers relies on the accuracy and lead-time provided by a flood warning system and if no system were installed, the probability of insufficient time will likely be 1 and the temporary flood barrier rendered ineffective. In many cases, combinations of measures reduce the probability of failures in the system by increasing redundancy. For example, mobile or temporary measures, while inexpensive, often have a high probability of failure unless they are applied tandem with a warning system. In contrast, other innovative alternatives, like the green roof, have a low failure probability, but high initial investment cost.

It is important to note that the application and evaluation of combinations of measures within the risk-based approach becomes increasingly complex dependent on number of interventions and the interdependence between the probability of success for any one intervention. An analysis of the entire risk reduction system would be required to accurately assess the performance of combinations of measures. These assessments require detailed information about the hazards and the performance of every innovation. Decision support tools that allow end-users to quickly evaluate different options can aid in these assessments (Zanuttigh et al. 2014), and a common set of performance indicators greatly reduces the complexity of the analysis.

Each performance indicator provides a necessary piece of information required to perform the described economic evaluation, as proposed in this chapter. A practical guideline is given for this economic evaluation, depending on whether an innovation aims to reduce the probability (Equation 5.5) or consequences (Equation 5.6) of flooding of a specific area. The practical examples have shown how each indicator is quantified and serve as an example of the use of the framework. However, the examples do not cover all types of analyses or tools that are available to quantify each indicator. It remains the innovators responsibility to determine which methods and tools should be used for this purpose.

The performance assessment used in this example assumes a discrete situation where the probability of failure, which is included as part of the Reliability indicator, does not depend on the load level (or flow velocity) and the level of damage is constant. In more detailed assessments, this dependency should be considered. In addition, due to the low frequency of extreme events, experience with the actual behavior of flood adaptation innovations is often lacking, resulting in uncertainties about their effectiveness and reliability. To address this issue, we encourage performing tests in laboratory and operational environments. Practical tests will help to reduce uncertainties, optimize the design, increase the reliability, while also providing insight in to ways an innovation interacts with an existing risk reduction system. Examples are tests of temporary flood defences in a test basin, or pilots with green roofs in cities.

Besides an economic analysis, the evaluation of flood risk reduction strategies, may also need to consider other impacts that cannot be easily translated into monetary terms, for example, an innovation's impact on ecology or nature or its societal impact. These impacts may be positive (e.g., building with nature interventions) or negative (e.g., loss of spatial views due to raising of flood defences) depending on the reference situation. End-users might also set additional conditions to innovative measures, such as limitations to the probability of operator error, logistical failure or an intended lifetime. Often, these criteria are difficult to assess in a laboratory environment, making it difficult to break into a new market or convince end-users that a technology is proven. Therefore, assessing the technical performance of innovations should be part of a broader assessment that also considers other impacts and end-user conditions.

Finally, the framework presented herein assumes that innovations are only evaluated based on their costs and benefits (i.e., risk reduction) from an economic point of view. While these economic analyses generally show that preventative structures (e.g., levees and barriers) are more cost-effective over the long term, flood adaptation innovations can provide an interim solution over the short term. Moreover, flood adaptation innovations can also be applied as secondary and tertiary measures aimed to further reduce risk for specific infrastructure (e.g., hospitals, railways or highways) and/or loss of life.

5.7 Concluding remarks

In this chapter, we developed a framework that enables end-users to evaluate and compare the performance of flood adaptation innovations through a common set of performance indicators. This framework aims to overcome the existing knowledge gap between the information that end-users require when evaluating whether to implement an innovative solution in a given system and the product-testing performed by innovators. To overcome this gap, we proposed a framework that can be used to evaluate any innovations' performance through a common set of indicators: effectiveness, durability, reliability and costs. These indicators allow for a calculation of cost and benefit over an innovations' lifetime, with the benefits expressed as the avoided flood damages. Ultimately, this allows end-users to compare innovations based on their benefit/cost ratio within a given implementation context.

Three examples were used to demonstrate how the framework can be used to obtain initial estimates the performance of every indicator, providing insight into an innovation's risk reduction and reliability, and allowing end-users to compare and choose between different innovations. This illustrates how different categories of flood adaptation innovations can be assessed using a standardized framework. While limited to flood adaptations in this chapter, the framework can be easily adjusted to be used to assess innovations intended to reduce risks associated with other climate related hazards such as extreme weather, droughts and wildfires.

6

Concluding remarks

This dissertation aimed at methods for evaluating flood risk reduction measures, specifically to assess the reliability of canal levees, to evaluate the effectiveness of emergency measures, to optimize the elevation levels of land fills and flood defences, and to assess the performance of flood adaptation innovations.

Each specific topic has been discussed in a separate chapter that ended with detailed conclusions and directions for further research for that specific topic. This final chapter states the overall conclusion of this dissertation (Section 6.1), followed by a summary of the detailed findings of each chapter (Section 6.2). Section 6.3 follows with recommendations (or directions) for scientific research and engineering practice. Finally, this dissertation ends with closing words (Section 6.4).

6.1 Main findings

This dissertation has advanced the risk-based approach for flood defences by developing methods that are able to assess the risk and/or reliability of specific interventions within a flood risk reduction system more accurately. Specifically, a method was developed that is able to quantify the probability of failure of canal levees (i.e., levees that align drainage canals in polders), taking in to account multiple loads acting on these levees. Additionally, a method is developed that assesses the reliability of emergency measures for flood prevention (e.g., sand bags), taking in to account human and logistical failure. This framework allows for the evaluation of the (cost-) effectiveness of emergency measures.

Furthermore, a method was developed that optimizes the elevation levels of land fills and flood defences depending on the size of the area to be protected, its land use and corresponding value. Finally, the insights gained in this dissertation were used to develop a method to assess the performance of flood adaptation innovations (i.e., solutions that have not been assessed in terms of risk reduction or solutions that have not yet been applied in practice).

Each separate chapter included case studies and/or practical examples of how methods can be used by decision makers to substantiate decisions regarding flood risk reduction. By doing so, the methods and examples in this dissertation aid decision makers in substantiating decisions regarding flood risk reduction. Ultimately, this dissertation has demonstrated how the risk-based approach, however complex it may seem, provides a better understanding of the uncertainty in the physical system and how it performs, and provides insight in the effectiveness and performance of several flood risk reduction interventions. This allows decision makers to make risk-informed decisions.

6.2 Detailed findings

6.2.1 Quantifying the failure probability of canal levees

An approach was developed to quantify the probability of failure of a canal levee, based on the probabilistic methods developed for riverine and coastal flood defences. The method has derived statistical models to take uncertainty of loads acting on canal levees in to account. The continuous probability density functions of these load variables were discretized in a predefined set of plausible levels with corresponding probability density. The total law of probability was used to account for i) regulation (and drainstop) of water levels in canals, ii) maintenance dredging and its influence on the hydraulic resistance of the canal, iii) the uncertain presence of traffic loads and iv) the uncertainty of the phreatic surface. In addition, reliability updating was used to demonstrate the impact of incorporating performance observations for the piping failure mechanism.

The method was applied to a case study of a canal levee system in the Netherlands. The probability of three failure mechanisms was determined: overflow, piping and instability. The probability of overflow was dominated by the probability of drainstop failure (i.e., failure to regulate water levels). For piping, the probability of failure was largely influenced by the (probability of) intrusion resistance of the bottom layer of the canal. A posterior analysis demonstrated the ability to reduce the probability of piping using performance observations (i.e., survived loads).

The probability of instability of the inner slope was dominated by the uncertainty in the phreatic surface and traffic loads. While this specific case study showed that the probability of horizontal sliding of the canal levee was small compared to other failure probabilities, this failure mechanism remains important to consider in levees subject to droughts. Overall, the probability of failure of the considered dike section was governed by the probability of instability and partly by piping, while the probability of overflow was negligible. These results were specifically found for the considered case study and demonstrated the ability of the method to accurately determine the failure probability of canal levees given the data available.

With insight in failure probabilities of canal levees, it is possible to assess risks associated with entire canal levee systems and to evaluate the effectiveness (in terms of reducing risk, either by reducing probability or consequences) of interventions in the system. The implementation of flood risk reduction measures (e.g., levee reinforcement or increasing drainage capacity) can also be prioritized based on their costs and benefits (or risk reduction).

6.2.2 Evaluating emergency measures for flood prevention

A method has been developed to determine the probability of failure of emergency measures to prevent flooding. The method is based on an event tree and includes three phases that need to be completed successfully for emergency measures to be effective: detection, placement and construction. The probability of failure of the detection and placement phase depend on human actions and logistics (i.e., if the measure is placed on time), while the probability of failure of the construction mode depends on the structural integrity of the emergency measure applied. To determine the probability of each mode, human error rates were determined and models for assessing the probability of logistical failure were derived. Finally, probabilistic models were used to assess the structural failure probability of an emergency measure.

This method was applied to a case study at a water board in the Netherlands, where the probability of failure of a flood defence system along a river was determined, including emergency measures for overtopping and piping. The probability of failure of overtopping emergency measures was found to be approximately 1/10 per measure, while the probability of failure of piping measures was found to be approximately 1/3 per measure. The probability of failure of both emergency measures was largely influenced by the probability of human errors during detection and placement and the probability of logistical failure (i.e., insufficient time for installing the emergency measure).

Ultimately, the probability of failure of the considered dike ring was reduced by a factor 4 for overtopping and a factor 2 for piping, after taking length effects in to account. The extent to which the probability of failure of a flood defence system can be increased strongly depends on the probability of human and logistical failure, the considered failure mode of the flood defence and the required length of the emergency measure. Using cost benefit analysis, the cost-effectiveness of emergency measures was compared with the cost-effectiveness of dike reinforcements as a flood risk reduction strategy. This analysis demonstrated that, for high initial failure probabilities, emergency measures are (far) less cost-effective than permanent reinforcements, making the latter preferred from an economical point of view. However, emergency measures can play a role as an interim solution to reduce flood risks, while permanent reinforcements are being prepared and realised. An advantage remains that the majority of the costs for emergency measures are only made when they are applied.

6.2.3 Optimizing portfolios of risk reduction strategies: flood defences and/or fills

A method was developed that optimizes the elevation levels of fills and flood defences. The method builds on existing methods for optimizing the total cost of flood risk reduction strategies. The model derived for land fills also represents the total costs for raising or floodproofing (existing) structures. Variations in the size of the area to be developed, its land use and corresponding value are included to model the total costs more accurately.

The optimal elevation levels were solved, providing insight in the drivers that determine the optimal elevation levels. The derived equations allow for optimization of the total cost of a single strategy (i.e., a flood defence or a fill), and combination of interventions (i.e., a fill behind an existing flood defence). Within the context of this economic model, we conclude that a system of flood defences is more economical than a land fill for larger areas (above an identified transition level). Fills are preferred for specific combinations of areas and land uses, or when low safety levels are required. The ratio between the marginal cost of fills and flood defences largely determines the size of the area for which flood defences become more economical. An increase of the marginal cost of fills leads to a reduction of its application range from an economic point of view (and vice versa).

Using these equations, several practical examples of decision problems in flood risk management have been elaborated and implications for developing and developed countries have been discussed. Overall, the practical examples demonstrate that investing in a single protective layer (fills or flood defences) is generally more economical than combining multiple protective layers (fills behind flood defences). Nevertheless, combinations of interventions can be attractive for specific cases. An example of such an optimal multi-layered strategy is found, when the value protected by the flood defence is low (agriculture) and the value protected by the fill is high (human lives/housing/ industry) or when the high value development is relatively small in size. In this case it makes sense to develop on raised land fills behind flood defences. The proposed approach also procides insight in tipping points between optimal strategies and the sensitivity for the main problem characteristics.

The derived methods have focused on land fills but can also be applied to similar strategies such as floodproofing of structures or raising houses, and is relevant for different areas subject to flood risks around the world (e.g., the Vietnam deltas or Japan coasts). Besides economic optimization of strategies, local requirements (limited rainfall flooding or settlements) or other drivers (e.g., time or budget constraints) may influence decision makers in deciding between different strategies.

6.2.4 Assessing the performance of flood adaptation innovations

A framework was proposed that allows end-users and innovators to evaluate the performance of flood adaptation innovations through a common set of performance indicators: effectiveness, durability, reliability and costs. These indicators allow for a calculation of cost and benefit over an innovations' lifetime, with the benefits expressed as the avoided flood damages. Ultimately, this allows end-users to compare innovations based on their benefit/cost ratio within a given implementation context.

Three examples were used to demonstrate how the framework can be used to obtain initial estimates the performance of every indicator, providing insight into an innovation's risk reduction and reliability, and allowing end-users to compare and choose between different innovations. This illustrates how different categories of flood adaptation innovations can be assessed using a standardized framework. While limited to flood adaptations in this chapter, the framework can be easily adjusted to be used to assess innovations intended to reduce risks associated with other climate related hazards such as extreme weather, droughts and wildfires.

6.3 Recommendations

6.3.1 For scientific research

With respect to quantifying the failure probability of canal levees: As the uncertainty of the phreatic surface proved to be dominant for the probability of failure of canal levees, I recommend studying the response of the phreatic surface to different forcing scenarios (e.g., heavy precipitation) with corresponding probability. Additionally, it is recommended to further investigate dependencies between canal water levels and the phreatic surface, considering canal system size and capacity, and the potential of incorporating performance observations in the quantification of the probability of instability of the inner slope. This could be done for hypothetical case studies, to gain insight in the potential effectiveness of Bayesian updating. Another possibility for further study is to use test loadings (e.g., by artificially increasing the phreatic surface and/or canal level) on existing levee sections to assess their performance under design conditions.

With respect to evaluating the effectiveness of emergency measures: The method developed in this dissertation was tested and validated partly in one case study for a water board in the Netherlands. The assignment of human error rates provided the best estimate given the current insights in human error probabilities, however, it is recommended to concentrate further work on validation of the human error rates and logistical failure rates, also considering organizational capacity for implementation of emergency measures. Performing more case studies or practical exercises can further substantiate the results. Although the structural failure probability of emergency measures remains negligible compared to human failure probabilities, I encourage further investigating the structural stability and integrity of proposed emergency measures, also considering possible length effects. These studies will also provide insight in the effectiveness of different types of emergency measures and will help to decide which can be applied once floods do occur. Finally, while this dissertation considered piping and overtopping emergency measures along river flood defence systems, it is recommended to apply the method to other failure modes and flood defences to study the potential effectiveness of emergency measures for these failure modes.

With respect to optimizing flood risk reduction strategies: As the models derived in this dissertation neglected time dependencies, economic growth and sea-level rise, I recommend including these aspects in follow up research of optimizing elevation levels of fills and flood defences. Further additions to the optimization model can include more detailed modelling of indirect damages (i.e., business losses) and loss of life. In addition, follow-up work can focus on the proportions of dimensionless quantities, to understand how preferred solutions depend on (and vary) depending on the chosen parameter values.

Regarding the design of flood risk reduction systems, I encourage analysing how other hazards (e.g., extreme winds, landslides or earthquakes) may influence the optimization models and elevation levels of the considered interventions in this dissertation. The inclusion of other hazards, or the possibility of simultaneous occurrence of several hazards (e.g., flooding and extreme winds), may give other results, because interventions used for one hazard may affect the effectiveness for other hazards. For example, the optimal elevation level of fills (or structures) may be lower if damage due to wind hazards are also taken in to account.

With respect to assessing the performance of flood adaptation innovations: Due to the low frequency of extreme events, experience with the actual behaviour of flood adaptation innovations is often lacking, resulting in uncertainties about their effectiveness and reliability. To address this issue, I encourage performing tests in laboratory and operational environments simulating design conditions. Additionally, the existing method can be expanded to allow for the evaluation of combinations of innovations. Another recommendation entails the application of the method developed for flood adaptation innovations to innovations for other natural hazards than considered in this dissertation. Examples include wind forcing (e.g., during tropical cyclones), wildfires and earthquakes.

6.3.2 For engineering practice and policy

Generally, practitioners in hydraulic engineering and flood risk management are encouraged to include risk-based approaches in their work. Specifically, the following recommendations are made:

With respect to quantifying the failure probability of canal levees: Based on the case study, this work found that the probability of overflow (i.e., insufficient height) of canal levees was negligible compared to the probability of instability, mainly because water levels in canal systems are regulated. Practitioners are encouraged to concentrate research of canal levees on the instability failure mode, under influence of the uncertainty of the phreatic surface inside the levee and possibility of traffic loads. Incorporating reliability updating in the analysis of the probability of instability can give sharper results, as was shown for the piping mechanism, especially if test loadings are used to assess the performance of the levee under design conditions. Ultimately, coupling target reliabilities of canal levees to the phreatic surface, instead of the canal water level, may be more effective. At least for levees comparable to those considered in the case study of this dissertation. Similarly, test loadings can be used to determine if piping can occur, by combining extreme water levels with reduced hydraulic resistance (due to dredging). Finally, I recommend using reliability and risk assessments in the assessments of canal levees, because this opens opportunities for evaluating the cost-effectiveness of non-structural measures in the system.

With respect to evaluating the effectiveness of emergency measures: Based on the analysis of emergency measures, I recommend considering them as an interim solution for flood defences with high initial failure probabilities, with known weak spots, until the construction or reinforcement of more permanent (structural) measures is realised. Moreover, given that the reliability of emergency measures largely depends on logistics, it is recommended to only consider emergency measures in systems with long lead times before hazard arrival (e.g., river systems in the Netherlands). Supposing practitioners still consider the use of emergency measures, I recommend concentrating efforts to improve their reliability on reducing the probability of human error and logistical failure with training and more detailed procedures and plans. Given that human error and logistical failure during the detection and placement phase are dominant, because these generally have failure probabilities in the order of 1/10 to 1/100 per use, it seems unwise to reduce the probability of structural failure to values below 1/100 per use.

With respect to optimizing flood risk reduction: In addition, with rising sea-levels, the question of how to effectively reduce flood risks will continue to challenge practitioners. Decision makers are encouraged to (continue) using risk-based approaches to analyse the existing system and in the design of new flood risk reduction systems. As was shown in this dissertation, based on a risk-based approach, insight into effective risk reduction strategies can be gained with relatively simple analyses. Further localized studies may consider a broader set of interventions (e.g., storm surge barriers, nourishments, reefs and different forms of damage reduction) and local inputs for elevation, damage density and investment costs. These additions will likely require numerical elaboration.

With respect to assessing the performance of flood adaptation innovations: To allow decision makers to make risk-informed decisions, innovators are encouraged to evaluate their innovations based on the four performance indicators developed in this dissertation. This requires innovators to expose all vulnerable parts of their innovations, putting *all cards on the table.* While this may not be preferred, in flood risk management, insight in the vulnerabilities of interventions is required to evaluate the reliability and risk reduction potential of any intervention. By exposing these vulnerabilities, innovators and decision makers can work together to optimize innovative solutions. Finally, to further facilitate the uptake of innovations, innovators and end-users are encouraged to develop standardized tools and questionnaires that help innovators through the screening process of an end-user.

6.4 Closing words

The principles and methods developed and applied in this dissertation may be used for challenges and applications in flood risk management, which have not been described specifically in the respective chapters. For example, the principles and methods for quantifying the failure probability of canal levees could also be applied dams, which like canal levees, are subject to rather constant hydraulic loads on a daily basis. In addition, methods for analysing the reliability and effectiveness of emergency measures for flood prevention can be applied to areas outside the Netherlands (e.g., California), where these measures are commonly applied as part of flood fighting. Those same methods can also be used to assess effectiveness of emergency measures for other hazards that include the detection and placement phases, such as wildfires or extreme weather. Similarly, methods for assessing the performance of flood adaptation innovations can also be used to assess the performance of innovations for different types of hazards. Finally, the methods for optimizing flood risk reduction strategies can also be applied to different floodproofing measures, such as raising houses, and is relevant for different areas subject to flood risks around the world (e.g., the Vietnam deltas or Japan coasts).

While the risk-based approach is widely recognized, applied and valued in the field of flood risk management in the Netherlands, its understanding and application internationally is often limited to high-level analyses. One reason may be that the detailed probabilistic risk-based methods (as applied in the Netherlands) require large amounts of detailed monitoring data (e.g., of water levels and subsoil) for statistical analysis, which may not always be justified economically or simply are too expensive. The practical examples and case studies in this dissertation demonstrate that in absence of detailed data, best estimates can be used in existing models or tools to provide insight in orders of magnitude of reliability and risk. Decision makers often require just that: exposing areas subject to high risk and interventions that are effective in reducing these risks.

Besides the cost-effectiveness of flood risk reduction strategies, other uncertainties, goals or aspects are getting more and more attention and may influence decision makers when considering flood risk reduction strategies. Examples include sustainability and limiting impact on ecology, but also the financial or governmental context. These aspects can result in a shift of preferred solutions away from the traditional (structural) solutions, which are often more economical (and have laid the basis for safety standards in the Netherlands for decades). Given these considerations, this dissertation recommends continuing the use of risk-based methods to assess the costs and benefits associated with every solution in monetary terms. At least to provide insight in their economic consequences. If insight in the reliability and risk of specific interventions is lacking, or if future aspects such as the financial or governmental context are uncertain, high level analyses can be used or developed to quantify reliability and risk, while engineering judgement can provide first estimates of input parameters. Ultimately, it is the responsibility of engineers and consultants to provide decision makers with complete and objective information, for them to make risk-informed decisions concerning flood risk reduction.

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Appendices

A

Parameters for canal levee piping case study

This appendix contains the values and distributions of general input parameters used in the probabilistic calculations in the case study.

Variable	**Parameter**	**Unit**	**Distribution**	**Mean (μ)**	**Coefficient of Variation (CV)**
Dry volumetric weight sand	y_s	kN/m^3	Deterministic	26.5	-
Wet volumetric weight sand	γ_{nat}	kN/m^3	Normal	17	0.05
Volumetric weight water	γ_w	kN/m^3	Deterministic	10	-
Permeability	K	m/s	LogNormal	$0.12 \cdot 10^{-4}$	1
Particle diameter top 70 % of subsoil	d70	M	LogNormal	$1.1 \cdot 10^{-8}$	0.15
Reference value for d70	$d70_m$	M	Deterministic	$2.08 \cdot 10^{-4}$	-
Angle of friction	θ_0	°	Deterministic	37	-
White's constant	H	-	Deterministic	0.25	-
Kinematic viscosity	v	m^2/s	Deterministic	$1.33 \cdot 10^{-6}$	-
Thickness of aquifer	D_1	m	LogNormal	1	0.1
Seepage length	L	m	LogNormal	22	0.1

Table 10: General input parameters piping section 4

B

Risk integrals of fills and polders

This appendix derives the risk integrals for the fill and the polder.

B.1 Integral of risk of a fill

Flood risks of an area on a fill are found with the following equation:

$$R_m = \frac{\int \frac{e^{-\frac{h_w-a}{b}}}{b} \cdot A \cdot V \cdot \frac{h_w-h_m}{d_{max}} \cdot dh_w}{r} \qquad \textit{for } h_m < h_w < h_m + d_{max}$$

$$R_m = \frac{\int \frac{e^{-\frac{h_w-a}{b}}}{b} \cdot A \cdot V \, dh_w}{r} \qquad \textit{for } h_w > h_m + d_{max} \qquad (B.1)$$

The following equation is found after filling in the boundaries of the integral.

$$R_m = \frac{1}{r} \cdot \left[\int_{h_m}^{h_m+d_{max}} \frac{e^{-\frac{h_w-a}{b}}}{b} \cdot A \cdot V \cdot \frac{h_w-h_m}{d_{max}} \, dh_w + \int_{h_m+d_{max}}^{\infty} \frac{e^{-\frac{h_w-a}{b}}}{b} \cdot A \cdot V \, dh_w \right] \qquad (B.2)$$

This integral simplifies to:

$$R_m = \frac{A \cdot V}{r} \cdot \left[\int_{h_m}^{h_m+d_{max}} \frac{e^{-\frac{h_w-a}{b}}}{b} \cdot \frac{h_w-h_m}{d_{max}} \, dh_w + \int_{h_m+d_{max}}^{\infty} \frac{e^{-\frac{h_w-a}{b}}}{b} \cdot dh_w \right] \qquad (B.3)$$

The left side of Eq. B.3 is solved as follows:

$$\int_{h_m}^{h_m+d_{max}} \frac{e^{-\frac{h_w-a}{b}}}{b} \cdot \frac{h_w-h_m}{d_{max}}\, dh_w = \frac{1}{d_{max}} \cdot \left(\int_{h_m}^{h_m+d_{max}} e^{-\frac{h_w-a}{b}} \cdot \frac{h_w}{b}\, dh_w - \int_{h_m}^{h_m+d_{max}} e^{-\frac{h_w-a}{b}} \cdot \frac{h_m}{b}\, dh_w \right)$$

And simplifies to:

$$\frac{1}{d_{max}} \cdot \left(\left[(-h_w - b) \cdot e^{-\frac{h_w-a}{b}} \right]_{h_m}^{h_m+d_{max}} - \left[h_m \cdot -e^{-\frac{h_w-a}{b}} \right]_{h_m}^{h_m+d_{max}} \right) \qquad \textit{(B.4)}$$

The right side of Eq. B.3 is solved as follows:

$$\int_{h_m+d_{max}}^{\infty} \frac{e^{-\frac{h_w-a}{b}}}{b} \cdot dh_w = \left[-e^{-\frac{h_w-a}{b}} \right]_{h_m+d_{max}}^{\infty} \qquad \textit{(B.5)}$$

Inserting Eq. B.4 and B.5 into Eq. B.3 gives:

$$R_m = \frac{A \cdot V}{r} \cdot \left(\frac{1}{d_{max}} \cdot \left\{ \left[(-h_w - b) \cdot e^{-\frac{h_w-a}{b}} \right]_{h_m}^{h_m+d_{max}} - \left[h_m \cdot -e^{-\frac{h_w-a}{b}} \right]_{h_m}^{h_m+d_{max}} \right\} + \left[-e^{-\frac{h_w-a}{b}} \right]_{h_m+d_{max}}^{\infty} \right)$$

After filling in the boundaries and simplifying, the following equation is obtained:

$$R_m = \frac{A \cdot V}{r} \cdot \left(\frac{1}{d_{max}} \cdot \left\{ [-h_m - d_{max} - b + h_m] \cdot e^{-\frac{h_m+d_{max}-a}{b}} - (-h_m - b + h_m) \cdot e^{-\frac{h_m-a}{b}} \right\} + e^{-\frac{h_m+d_{max}-a}{b}} \right) =$$

$$R_m = \frac{A \cdot V}{r \cdot d_{max}} \cdot \left(b \cdot e^{-\frac{h_m-a}{b}} - d_{max} \cdot e^{-\frac{h_m+d_{max}-a}{b}} - b \cdot e^{-\frac{h_m+d_{max}-a}{b}} + d_{max} \cdot e^{-\frac{h_m+d_{max}-a}{b}} \right) =$$

$$R_m = \frac{A \cdot V \cdot b}{r \cdot d_{max}} \cdot \left(e^{-\frac{h_m-a}{b}} - e^{-\frac{h_m+d_{max}-a}{b}} \right) \qquad \textit{(B.6)}$$

B.2 Integral of risk of a circular polder.

Flood risks of an area surrounded by flood defences are found with the following equation:

$$R_d = \frac{\int \frac{e^{-\frac{h_w-a}{b}}}{b} \cdot A \cdot V \cdot \frac{h_w-h_0}{d_{max}} \cdot dh_w}{r} \qquad \textit{for } h_d < h_w < h_0+d_{max}$$

$$R_d = \frac{\int \frac{e^{-\frac{h_w-a}{b}}}{b} \cdot A \cdot V\, dh_w}{r} \qquad \textit{for } h_w > h_0+d_{max} \qquad \textit{(B.7)}$$

The following equation is found after filling in the boundaries of the integral.

$$R_d = \frac{1}{r} \cdot \left[\int_{h_d}^{h_0+d_{max}} \frac{e^{-\frac{h_w-a}{b}}}{b} \cdot A \cdot V \cdot \frac{h_w-h_0}{d_{max}}\, dh_w + \int_{h_0+d_{max}}^{\infty} \frac{e^{-\frac{h_w-a}{b}}}{b} \cdot A \cdot V\, dh_w \right] \qquad \textit{(B.8)}$$

This integral simplifies to:

$$R_m = \frac{A \cdot V}{r} \cdot \left[\int_{h_d}^{h_0+d_{max}} \frac{e^{-\frac{h_w-a}{b}}}{b} \cdot \frac{h_w-h_0}{d_{max}}\, dh_w + \int_{h_0+d_{max}}^{\infty} \frac{e^{-\frac{h_w-a}{b}}}{b} \cdot dh_w\right] \quad (B.9)$$

The left side of Eq. B.9 is solved as follows:

$$\int_{h_d}^{h_0+d_{max}} \frac{e^{-\frac{h_w-a}{b}}}{b} \cdot \frac{h_w-h_0}{d_{max}}\, dh_w = \frac{1}{d_{max}} \cdot \left(\int_{h_d}^{h_0+d_{max}} e^{-\frac{h_w-a}{b}} \cdot \frac{h_w}{b}\, dh_w - \int_{d}^{h_0+d_{max}} e^{-\frac{h_w-a}{b}} \cdot \frac{h_0}{b}\, dh_w\right)$$

And simplifies to:

$$\frac{1}{d_{max}} \cdot \left([\,(-h_w-b) \cdot e^{-\frac{h_w-a}{b}}]_{h_d}^{h_0+d_{max}} - [\,h_0 \cdot -e^{-\frac{h_w-a}{b}}]_{h_d}^{h_0+d_{max}}\right) \quad (B.10)$$

The right side of Eq. B.9 is solved as follows:

$$\int_{h_0+d_{max}}^{\infty} \frac{e^{-\frac{h_w-a}{b}}}{b} \cdot dh_w = [\,-e^{-\frac{h_w-a}{b}}]_{h_0+d_{max}}^{\infty} \quad (B.11)$$

Inserting Eq. B.10 and B.11 into Eq. B.9 gives:

$$R_d = \frac{A \cdot V}{r} \cdot (\frac{1}{d_{max}} \cdot \{(-h_w-b) \cdot e^{-\frac{h_w-a}{b}}]_{h_d}^{h_0+d_{max}} - [\,h_0 \cdot -e^{-\frac{h_w-a}{b}}]_{h_d}^{h_0+d_{max}}\} + [\,-e^{-\frac{h_w-a}{b}}]_{h_0+d_{max}}^{\infty})$$

After filling in the boundaries and simplifying, the following equation is obtained:

$$R_d = \frac{A \cdot V}{r} \cdot \left(\frac{1}{d_{max}} \cdot \left\{[-h_0 - d_{max} - b + h_0] \cdot e^{-\frac{h_0+d_{max}-a}{b}} - (-h_d - b + h_0) \cdot e^{-\frac{h_d-a}{b}}\right\} + e^{-\frac{h_0+d_{max}-a}{b}}\right) =$$

$$R_d = \frac{A \cdot V}{r \cdot d_{max}} \cdot \left(-d_{max} \cdot e^{-\frac{h_0+d_{max}-a}{b}} - b \cdot e^{-\frac{h_0+d_{max}-a}{b}} + (h_d + b - h_0) \cdot e^{-\frac{h_d-a}{b}} + d_{max} \cdot e^{-\frac{h_0+d_{max}-a}{b}}\right) =$$

$$R_d = \frac{A \cdot V \cdot b}{r \cdot d_{max}} \cdot ([\frac{h_d}{b} - \frac{h_0}{b} + 1]e^{-\frac{h_d-a}{b}} - e^{-\frac{h_0+d_{max}-a}{b}}) \quad (B.12)$$

Eq. B.12 assumed that the flood defence level (h_d) is lower than the level where maximum flood damages occur ($h_d < h_0+d_{max}$). If the flood defence level is higher than this level, Eq. B.7 simplifies to Eq. B.13:

$$R_d = \frac{\int \frac{e^{-\frac{h_w-a}{b}}}{b} \cdot A \cdot V\, dh_w}{r} \qquad \text{for } h_d > h_0+d_{max} \quad (B.13)$$

The integral in Eq. B.13 is solved, for the lower (h_d) and upper (∞) boundaries as follows:

$$R_d = \frac{A \cdot V}{r} \cdot \int_{h_d}^{\infty} \frac{e^{-\frac{h_w-a}{b}}}{b} \cdot dh_w = R_d = \frac{A \cdot V}{r \cdot b} \cdot e^{-\frac{h_d-a}{b}} \quad (B.14)$$

C

Estimating the probability of failure of a temporary flood barrier

C.1 Probability of human error

According to these guidelines, an operator operating at a rule-based level can have an error probability ranging from $2{\cdot}10^{-4}$ to $2{\cdot}10^{-2}$ per task.

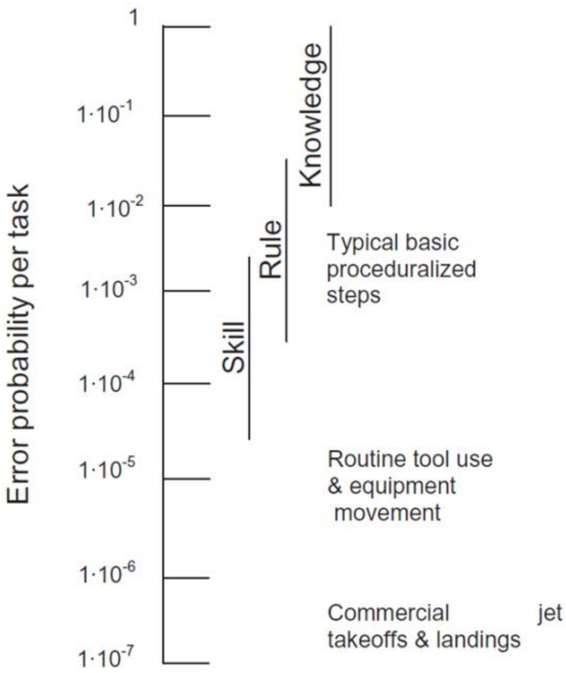

Figure 66: Human error probabilities and performance levels by Watson and Collins (Bea 2010)

C.2 Probability of sliding failure

Water-filled tubes can be seen as small gravity structures which obtain their stability through self-weight (W [kN/m]). The loads on the structure consist of the horizontal water pressure ($F_{w;h}$ [kN/m]) and upward water pressure ($F_{w;v}$) [kN/m]).

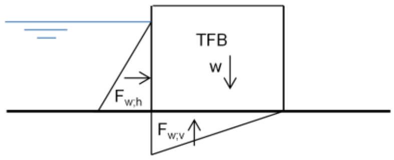

Figure 67: Typical loads on a schematized temporary water-retaining structure consisting of layers of sand bags.

These structures are subject to the following failure modes: overflow/overtopping, rotation instability, horizontal sliding, seepage and structural failure. The stability is largely influenced by the weight and the development of upward water pressure under the structure, which depends on the subsoil, loading time and connection between the structure and the subsoil (Lendering et al. 2016).

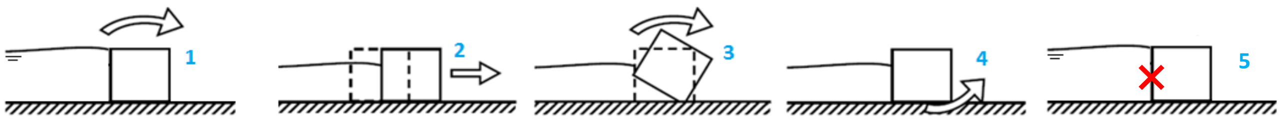

Figure 68: Typical structural failure mechanisms of temporary water retaining structures: Overflow (1), Sliding (2), Rotation (3), Seepage (4) and (5) Structural failure (Boon 2007)

In this example, we will demonstrate how the probability of failure is calculated for the sliding failure mechanism. Sliding is often governing for these structures and occurs when the horizontal force on the structure exceeds the friction force between the structure and the subsoil due to self-weight:

$$Fs_{sliding} = \frac{f \cdot (\mathrm{W} - \mathrm{F}_{w;v})}{F_{w;h}} > 1 \qquad (C.1)$$

Stability is obtained when the safety factor [$Fs_{sliding}$] is higher than one. The probability of technical failure is estimated with Monte Carlo Simulation, using the following input:

Variable	Parameter	Distribution	Equation	Value	Unit
Ø	Friction angle of subsoil (clay)	Normal	-	μ = 22.5 σ = 2	°
y_W	Volumetric weight water	Deterministic		10	kN/m^2
H	Water level inside structure	Normal	-	μ = 0.6 σ = 0.05	m
L	Length of structure	Deterministic	-	1.0	m
B	Width of structure	Deterministic	-	0.9	m
f	friction coefficient		tan(Ø)		-
V	Volume of structure	-	$B \cdot H_r \cdot L$	0.42	m3
$F_{w;v}$	Upward water pressure	-		0	kN/m
W	Weight of structure	-	$V \cdot y_w$	4.2	kN/m
H_w	Water level	-		0.58	m
$F_{w;h}$	Horizontal force	-	$0{,}5 \cdot y_w \cdot H_r^2$	1.25	kN/m
F_S	Safety factor	-	$W \cdot f / F_{w;h}$	1.0	-
P_f	Probability of failure (conditional on H_w)	-			1/50

Table 24: Input data for sliding failure probabilistic calculations

The estimated probability of structural failure, for a given water level of 0.58 meter, is 1/50 per use.

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Curriculum Vitae

20-04-1988	Born in Willemstad, Curaçao

Education

2010 – 2012	Master of Science in Civil Engineering, Delft University of Technology, Hydraulic Structures and Flood Risk Thesis: Polder Terminal: A Risk-Based Design Supervisor: Prof. drs.ir. J.K. Vrijling
2006 – 2010	Bachelor of Science in Civil Engineering, Delft University of Technology
2001 – 2006	Pre-University Secondary Education (VWO), Physical Sciences and Economy, Peter Stuyvesant College, Curaçao

Experience

2015 – present	Consultant Horvat & Partners, Delft, the Netherlands
2012 – present	Researcher Hydraulic Structures and Flood Risk, Delft University of Technology, the Netherlands
2010 – present	Structural Engineer LB Constructie, Curaçao
2012	Graduate Intern Royal HaskoningDHV, Rotterdam, the Netherlands
2011	Intern Superintendent PUMA (Van Oord), Rotterdam, the Netherlands
2008 – 2012	Student-assistant Design Courses Delft University of Technology, the Netherlands

Awards and nominations

2013	Nomination for ‘Waterbouwprijs 2012’
2006	Best Graduate of the Netherlands Antilles

List of publications

The following publications are related to this dissertation:

Peer-reviewed journals

Lendering, K. T., Jonkman, S. N., Van Gelder, P. H. A. J. M., & Peters, D. J. (2015). Risk-based optimization of land reclamation. *Reliability Engineering and System Safety*, *144*, 1–11. http://doi.org/10.1016/j.ress.2015.07.025

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Under review

***Lendering, K.T.**, Jonkman, S.N., van Ledden, M., Vrijling, J.K.,* Optimizing portfolios (of combinations) of risk reduction strategies: flood defences and/or land fills.

Conferences

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