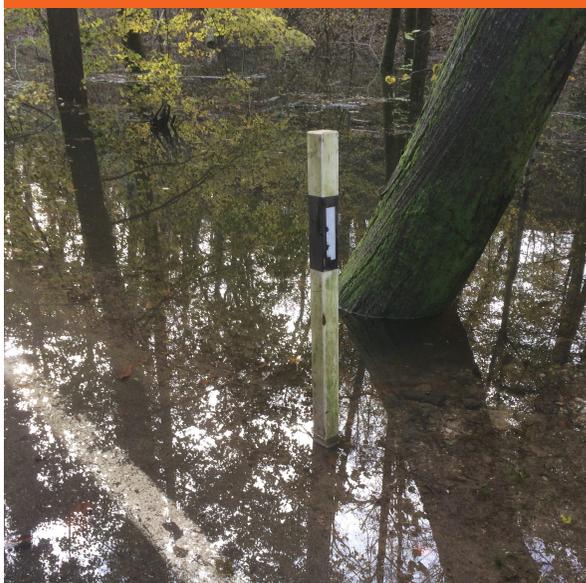


# Hierarchical Modelling of Flood Risk for Engineering Decision Analysis



Rocco Custer

PhD Thesis

Department of Civil Engineering  
2015

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PhD Thesis

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2015

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# Preface

This thesis is submitted as a partial fulfilment of the requirements for the Danish PhD degree. The work has been carried out at the Department of Civil Engineering at the Technical University of Denmark and took place in the period between January 2011 to December 2014, with Professor Michael H. Faber and Associate Professor Kazuyoshi Nishijima as supervisors. This thesis is partially based upon two academic articles (one under review at *Georisk*, one accepted by *Natural Hazards*).

Copenhagen, March 19<sup>th</sup> 2015.

Rocco Custer

# **Preface to the published version**

The thesis was defended at a public defence on the 18.06.2015. The official assessment committee was formed by:

- Assoc. Prof. Sebastian Thöns (chair of the committee), DTU
- Charles Baubion, OECD
- Prof. Sebastiaan N. Jonkman, TU Delft

Copenhagen, June 19<sup>th</sup> 2015.

Rocco Custer

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I want to thank my parents, brothers and my friends around the world for their support and friendship. Finally, I want to thank my girlfriend Veera, to whom I dedicate this thesis, for her understanding when I had to work evening and nights, for her support and sense of humour.

# Abstract

Societies around the world are faced with flood risk, prompting authorities and decision makers to manage risk to protect population and assets. With climate change, urbanisation and population growth, flood risk changes constantly, requiring flood risk management strategies that are flexible and robust. Traditional risk management solutions, e.g. dike construction, are not particularly flexible, as they are difficult to adapt to changing risk. Conversely, the recent concept of integrated flood risk management, entailing a combination of several structural and non-structural risk management measures, allows identifying flexible and robust flood risk management strategies. Based on it, this thesis investigates hierarchical flood protection systems, which encompass two, or more, hierarchically integrated flood protection structures on different spatial scales (e.g. dikes, local flood barriers and dry-proofed buildings), which jointly reduce risk. Hierarchical flood protection systems offer several advantages compared to single-structure flood protection systems, since they can be precision-tailored to fit risk reduction requirements and allow for flexible adaption of the protection system to changing flood risk.

In the presence of flood protection structures, flood development depends on the state of all protection structures in the system. As such, hazard is a function not only of rainfall and river discharge, but also of protection structures' fragility. A methodology for flood risk analysis and decision analysis for hierarchical flood protection systems is proposed, which allows for joint consideration of hazard models and fragility models of protection structures.

In the implementation of the flood risk analysis methodology several challenges are identified, two of which are addressed in the present thesis. First, design and optimisation of a hierarchical flood protection system generally entails decisions about structures at different spatial scales, which, in turn, may require risk assessment at different spatial resolutions and levels of detail. Consistent risk modelling at different spatial scales may therefore require up- and downscaling of data and models under due consideration of uncertainties and dependencies. In this thesis, a methodology is

proposed for spatially disaggregating an aggregated building portfolio considering disaggregation uncertainty and spatial correlation. The methodology is applied to the disaggregation of portfolios of buildings in two communes in Switzerland. The relevance of disaggregation uncertainty to natural hazard risk assessment is illustrated with a simple flood risk assessment example.

A second challenge - fragility and vulnerability modelling of all protection structures in the hierarchical flood protection system - is identified. To optimise the design of protection structures, fragility and vulnerability models must allow for consideration of decision alternatives. While such vulnerability models are available for large protection structures (e.g. dikes), engineering vulnerability models that allow considering the impact of flood proofing measures on residential building vulnerability seem to be lacking. Thus, a flood vulnerability modelling approach for residential buildings is proposed, which allows for detailed building and hazard characterisation and models damages through explicit consideration of damage processes. The modelling approach allows for describing the impact of flood proofing measures on building vulnerability and can be utilised as a basis for decision analysis.

The concept and usefulness of hierarchical flood protection systems, as well as the implementation of the flood risk analysis methodology and the vulnerability modelling approach are illustrated with an example application.

In summary, the present thesis provides a characterisation of hierarchical flood protection systems as well as several methodologies to model such systems. It aims at increasing understanding of hierarchical flood protection systems and provides modelling approaches to facilitate further research and the implementation of hierarchical flood protection systems in practice.

# Resumé

Samfundet verden over er konfronteret med risikoen for oversvømmelse, hvilket tvinger myndigheder og andre beslutningstagere til at håndtere konsekvenserne af oversvømmelser gennem risikostyring for at beskytte befolkningen og deres aktiver. Klimaændringer, urbaniseringen samt befolkningstilvæksten medfører at risikoen for oversvømmelse ændres konstant, hvilket kræver at strategierne for risikostyringen er fleksible og robuste. Traditionelle løsninger indenfor risikostyring såsom diger er ikke særlige fleksible, da de ikke umiddelbart kan tilpasses ændrede risiko niveauer. Omvendt er fleksible og robuste strategier for risikostyring nyligt blevet identificeret gennem konceptet om integreret risikostyring for oversvømmelser, hvilket indeholder en kombination af flere strukturelle og ikke-strukturelle metoder. Denne afhandling undersøger hierarkiske beskyttelsessystemer mod oversvømmelser, der indeholder to eller flere integreret hierarkiske strukturer til beskyttelse mod oversvømmelser. Disse er implementeret på forskellige niveauer f.eks. diger, lokale barrierer, bygninger designet til at kunne håndtere oversvømmelser og tilsammen vil disse reducere risikoen. Der er adskillige fordele ved hierarkiske beskyttelsessystemer sammenlignet med beskyttelsessystemer udelukkende bestående af en enkelt struktur. Herunder kan nævnes deres mulighed for at være præcist skræddersyet til den krævede risikoreduktion samt at de tillader tilpasning af de fleksible beskyttelsessystemer for skiftende risiko for oversvømmelse.

Ved tilstedeværelse af strukturelle barrierer mod oversvømmelser afhænger oversvømmelsens udvikling og konsekvenser af tilstanden af barriererne i systemet. Risikoen er ikke udelukkende funktion af nedbørsmængden og udledningsmængden fra floder, men også af skrøbeligheden af barriererne. En ny metode, til at analysere risikoen for oversvømmelser og beslutninger i forbindelse med hierarkiske beskyttelsessystemer og som giver mulighed for samlede overvejelser vedrørende risikomodeller og modelleringen af barrierernes skrøbelighed, er beskrevet

I implementeringen af metoden for risikoanalysen er der identificeret flere udfordringer, hvoraf to er behandlet i nærværende afhandling.

Først, design og optimering af et hierarkisk beskyttelsessystem for oversvømmelser indebærer generelt beslutninger om strukturer på forskellige rumlige skalaer. Disse kan til gengæld kræve risikovurdering niveauer. Konsekvent risiko modellering på forskellige rumlige skalaer kan kræve op- og nedskalering af data og modeller under hensyntagen til usikkerheder og afhængigheder. I denne afhandling er foreslået en metode for rumligt opdeling af en samlet portefølje af bygninger, under hensyntagen til usikkerheden ved opdelingen og den rumlige korrelation. Metoden anvendes på opdelingen af porteføljer for bygninger i to kommuner i Schweiz. Relevansen af at modellere usikkerheden ved opdelingen i risikovurderingen af naturkatastrofer er illustreret med et simpelt eksempel for vurderingen af risikoen for oversvømmelse.

Den anden identificerede udfordring er modelleringen af skrøbeligheden og sårbarheden af barriererne i det hierarkiske beskyttelsessystem mod oversvømmelser. For at optimere udformningen af barriererne, er det nødvendigt at modelleringen af disses skrøbelighed og sårbarhed giver mulighed for at betragte alternative design. Modeller for modelleringen af sårbarheden for større beskyttelsessystemer (f.eks. diger) er tilgængelige, hvorimod modeller for det medtager sårbarheden af lokale barrierer for beboelsesbygninger synes at mangle. I den nærværende afhandling foreslås en metode til modellering af beboelsesbygninger, der medtager detaljeret karakteristika og risikoniveau for bygningen samt eksplicit modellerer skadeudviklingen. Metoden tillader at barrierernes påvirkning på sårbarheden mod oversvømmelse af bygningen medtages og derved kan benyttes som grundlag i en beslutningsanalyse.

Konceptet og mulighederne med hierarkiske beskyttelsessystemer samt implementeringen af risikoanalysen for oversvømmelser og metoden for at modellere sårbarheden af barriererne er illustreret med et eksempel.

Kortfattet, tilvejebringer nærværende afhandling en karakteristik af hierarkiske beskyttelsessystemer mod oversvømmelser samt flere metoder til at modellere sådanne systemer. Dens målsætning er at øge forståelsen af hierarkiske beskyttelsessystemer og leverer modelleringsmetoder der kan bruges til fremme yderligere forskning og implementeringen af hierarkiske beskyttelsessystemer i praksis.



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# **1 Introduction**

This chapter provides relevant background information, highlighting the relevance of flood risk management and framing several inherent challenges. Further, the hierarchical flood protection system concept is introduced, along with objectives and scope of this thesis.

## 1.1 Background

Floods are caused by heavy rainfalls, the overflow of water bodies, or the overflow of water streams, which may have negative impacts on society, causing fatalities and economic damage. Flood risk is a measure to jointly characterise the probability of occurrence and the consequences of floods<sup>1</sup>; its understanding requires thorough knowledge of its two components.

According to EM-DAT (2014), over the last 30 years, floods have caused an average of 6'000 fatalities yearly and approximately 20 billion USD in economic damage yearly. According to Jha *et al.* (2012) flood is globally the most frequent natural hazard. EM-DAT (2014) and DFO (2014) indicate that flood events are registered on every continent and most countries, with a few exceptions for deserts and polar regions. Furthermore, according to EM-DAT (2014) the number of flood events meeting their reporting criteria<sup>2</sup> has significantly increased from the 1980s, as illustrated in Figure 1.1. Notwithstanding that event reporting may have improved, it is generally accepted that flood risk increased over the last decades and a further increase is expected in the future (Jha *et al.* 2012). The expected increase can be mainly attributed to three factors: climate change, population growth and urban development.

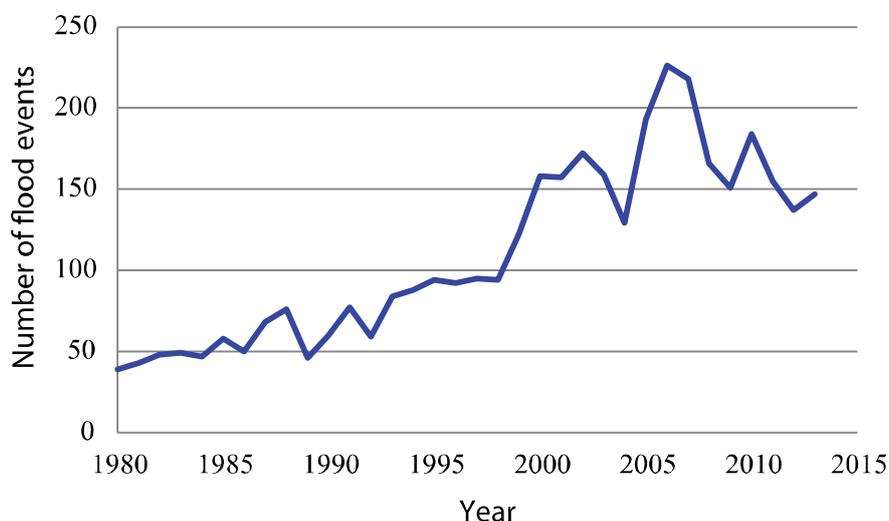


Figure 1.1 - Global flood events per year (Data source: EM-DAT 2014)

The effects of climate change are diverse and must be differentiated by geographical region. Nevertheless, the research community generally agrees that climate change will

<sup>1</sup> A formal definition of flood risk is provided in Section 3.2.

<sup>2</sup> The EM-DAT (2014) reporting criteria is the following: "...at least one of the following criteria must be fulfilled: 10 or more people reported killed, 100 or more people reported affected, declaration of a state of emergency or call for international assistance."

lead to more frequent and severe extreme meteorological events (IPCC 2014a). Thus, frequency and intensity of heavy rainfall events, as well as peak river discharge following heavy rainfall, are expected to increase in many parts of the world, leading to higher frequency of flooding. Climate change is also expected to cause a rise in sea levels, increasing the probability of flooding in low-lying coastal areas (Field *et al.* 2012).

Population growth and urban development both contribute to an increased societal loss potential. Population growth leads to more people and assets potentially exposed to floods; urban development leads to a concentration of population and assets in relatively small urbanised areas. According to Bouwer (2010), these two factors are responsible for greater future flood risk increase than climate change. Bouwer (2010) and Evans *et al.* (2006) indicate how serious the potential future increase in flood risk might be. The former calculates that flood risk in The Netherlands might increase between 96% and 716% from 2000 to 2040, whereas the latter calculates up to twentyfold increase in economic risk for some regions in the United Kingdom by the 2080s, if current spending for flood risk management is not increased. These projections are alarming, however they are subject to substantial uncertainties, see Hall and Solomatine (2008) and Bouwer (2010).

Given the substantial threat to population and societal assets, decision makers in supranational, national and regional organisations, as well as decision makers in commercial entities and private persons are asked to manage flood risk. Flood risk management entails the assessment of flood risk and, if necessary, the implementation of flood risk management measures to appropriately handle risk.

The importance of managing flood risk can be illustrated anecdotally by comparing the number of fatalities from flood events in developed and developing countries. On average, a flood event in developing countries causes almost ten times as many fatalities as an event in developed countries (EM-DAT 2014). There are many reasons for this difference, e.g. higher population density in several developing countries. However, the difference in flood risk management plays a significant role. Developed countries have been managing flood risk for over a century, while developing countries lack resources and, often, knowledge to appropriately manage risk; more fatalities are the consequence (Hallegate 2012).

Decision makers can often choose between a plethora of structural and non-structural flood risk management measures, with a range of spatial scales and investment costs (Jha *et al.* 2012). Structural measures include dams protecting a whole

valley, dikes protecting a flood plain, local flood barriers protecting a village and flood proofing measures protecting an individual building. Non-structural measures include early warning systems, population education to raise risk awareness, disaster management plans, risk transfer instruments and spatial planning.

Historically, flood risk management has been dominated by the construction of large flood protection structures, e.g. dikes. Their design was generally based on a design event (e.g. 100 year event) from which the structure ought to offer protection (Merz, Hall *et al.* 2010). Potential flood event consequences were not explicitly considered in the design process. Once the structure was built, in a false sense of security, little consideration was given to residual risk and the possibility that the protection structure might fail (Tobin 1995). In recent years, a more thorough approach has become the norm, considering all possible flood events and their consequences. With this shift, flood risk assessment has become the basis for identifying optimal flood risk management strategies<sup>3</sup> through risk-based decision analysis. In Europe and elsewhere, this shift in perspective has been accelerated by regulations requiring authorities to develop flood management plans on the basis of flood risk assessments (Bubeck 2013).

Identification of an appropriate flood risk management strategy should consider its economic costs and benefits, as well as performance of the strategy under a number of future flood risk scenarios (Field *et al.* 2012). The necessary compromise between cost efficiency and future sustainability of a solution is well captured by Revkin (2011): “*The basic issue is finding ways to build into near-term investments and choices an appropriate consideration of long-term trends and worst-case scenarios.*”. This balance is particularly difficult when designing large protection structures, as they entail substantial investments and long planning and construction periods. Once built, with a certain protection height, they offer little flexibility to adapt to changing flood risk. The recognition that large protection structures alone cannot adequately assure safety of population and assets has led to the development of the integrated flood risk management concept. Integrated flood risk management entails the combination of several flood risk management measures which jointly manage risk; for example, large protection structures are complemented by local flood barriers and early warning system measures. An extensive description of integrated flood risk management is found in Jha *et al.* (2012) and elsewhere in literature. For instance, Dawson *et al.* (2004b)

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<sup>3</sup> A flood risk management strategy is here understood as a comprehensive set of measures to understand and treat flood risk in a specific region.

recommend complementing large protection structure with non-structural measures such as spatial planning policies and risk transfer instruments. Bubeck (2013) and Joseph (2014) emphasise the importance of implementing risk reducing measures in private buildings to complement large scale protection structures.

The large variety of possible integrated flood risk management strategies makes the identification of the optimal flood risk management strategy challenging. Comprehensive science-based decision support for integrated flood risk management, with sustainable solutions as a goal and appropriate consideration of risk and uncertainties, is essential. This thesis contributes to this goal by characterising and modelling hierarchical flood protection systems, as introduced in the next section.

## 1.2 Hierarchical flood protection system

A hierarchical flood protection system entails two or more sequentially integrated structural risk management measures along the route of flood water from its source to affected persons and assets (see Figure 1.2 for a schematic illustration). In accordance with the principles of integrated flood risk management, in a hierarchical flood protection system flood risk is jointly managed by several risk management measures. While the bulk of risk reduction is still assured by large protection structures, such as dikes and dams, these are complemented by smaller structures, e.g. secondary dikes, local flood barriers or flood proofing measures applied to buildings to protect their interiors. Hierarchical flood protection systems have several advantages, some of which are outlined in the following.

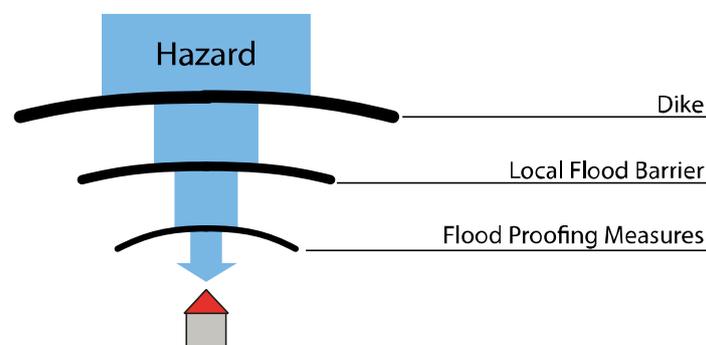


Figure 1.2 - Schematic representation of a hierarchical flood protection system with three hierarchy levels

When flood protection is assured by a single large protection structure, flooding probability is homogenously reduced for a whole flood plain, including villages, cities, agricultural areas and forests. Dike height is often dictated by the required safety level of urban areas; however, agricultural areas and forests on the flood plain profit from the same flood protection, which may not be economically justifiable and can thus be seen as a waste of societal resources. In contrast, a hierarchical flood protection system can be tailored to reduce flood risk in a more specific way: areas with high asset value density can be protected on a higher safety level than agricultural areas and forests, see Figure 1.3 for illustration. As a result, expected costs (including costs of constructing and maintaining a flood protection system, as well as expected flood consequences) entailed by a hierarchical flood protection system are generally lower than for a single-structure flood protection system.

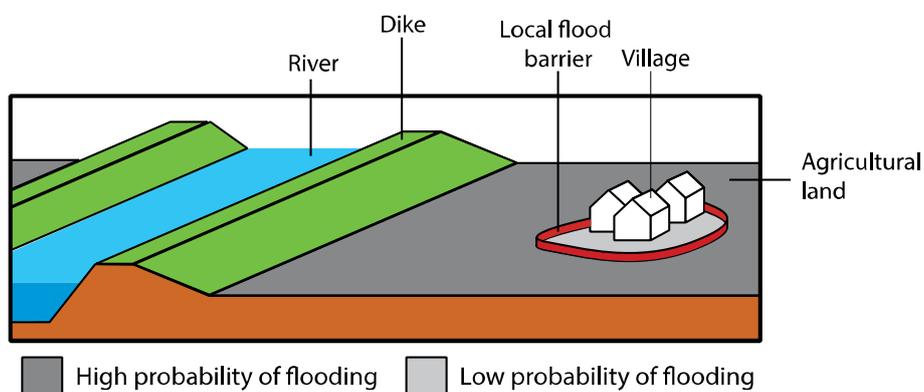


Figure 1.3 - Illustration of the effect of a hierarchical flood protection system on the probability of flooding

A further advantage of hierarchical flood protection systems emerges with the challenge faced by decision makers when planning large flood protection structures. Given the many uncertainties about future flood risk, such structures can easily be over-designed, resulting in wasted resources, or under-designed, resulting in insufficient protection. In such a situations, it is advantageous to identify flood risk management strategies that are flexible (adaptable for the future, if necessary) and robust (i.e. good performance in multiple future flood risk scenarios). The consideration of a hierarchical flood protection system allows for flexible adaption of the flood protection system, through upgrading the protection structures on a hierarchy level or adding a hierarchy level if, and when, required.

Given these considerations, hierarchical flood protection systems seem useful. Whereas hierarchical flood protection systems may naturally arise in practice, they are rarely actively considered when planning flood protection systems. Specifically, when planning large flood protection structures, the possibility of adding smaller complementary structures is often ignored. As a result, the flood protection system might be suboptimal and disregarding the aforementioned advantages. A possible reason for the little consideration of hierarchical flood protection systems in practice might be the challenges encountered in modelling such systems. In the following, three challenges are identified.

Hierarchical flood protection systems entail protection structures of different sizes and with different reach of protection. The consideration of such different flood protection structures may require flood risk to be assessed at different spatial scales, i.e. with different resolutions and levels of detail. Therefore, the first challenge pertains to consistent modelling of risk at different spatial scales, which can require data and models to be scaled up or down to consistently represent risk at each scale.

A second challenge is faced with the computation of flood scenarios with consideration of the state of all flood protection structures. When assessing flood risk for areas protected by protection structures, the possibility of protection structure breaches should be considered. Given that each protection structure can be breached at different locations, multiple different breaching scenarios must be modelled to appropriately assess flood risk. When breaches on several hierarchy levels are considered, the total number of breaching scenarios to be modelled grows exponentially with the number of hierarchy levels, and with it, computational costs increase rapidly. Several approaches are available to model flood with consideration of breaching scenarios of individual flood protection structures (e.g. Dawson and Hall 2006; Vorogushyn *et al.* 2010); however, these approaches must be extended to allow considering breaches on more than one hierarchy level, in a computationally efficient manner.

A third challenge pertains to the modelling of the vulnerability and fragility of all considered protection structures. Optimisation of a hierarchical flood protection system requires the joint optimisation of several individual protection structures, for each of which, decision alternatives need to be compared. Therefore, for every considered protection structure, a fragility or vulnerability model is required, which allows for consideration of decision alternatives. Fragility models are available for dikes, which are based on structural reliability theory and allow for structure optimisation for flood

risk management (see e.g. Voortman 2003). Little research is available on fragility of smaller protection structures, like local flood barriers. Vulnerability models for residential buildings are available, which allow considering the effect of flood proofing measures. However, no model providing sufficient modelling details for its practical application is available.

These challenges indicate that research work is necessary to facilitate the modelling of hierarchical flood protection systems.

### *1.3 Objective*

This thesis aims to contribute to the characterisation and modelling of hierarchical flood protection systems, as well as enabling decision optimisation within a hierarchical flood protection system. In particular, the following objectives are identified:

- Define and characterise hierarchical flood protection systems.
- Propose a modelling approach for flood risk analysis within hierarchical flood protection systems. Further, define a model for decision analysis and decision optimisation for hierarchical flood protection systems.
- Apply the concept of hierarchical flood protection systems to an example study area.
- Develop a flood vulnerability model for residential buildings that can be utilised as part of a decision analysis, i.e. a vulnerability model sensitive and detailed enough to represent vulnerability changes introduced by flood proofing measures.
- Make a contribution to the consistent modelling of flood risk at different spatial scales, particularly to the spatial disaggregation of portfolio data under consideration of uncertainties and dependencies.

#### *1.3.1 Scope of the thesis*

Flood risk assessment and management are broad and complex study fields, encompassing a number of disciplines, including meteorology, hydrology, geography and civil engineering. It is not possible to cover all relevant aspects in either a general or detailed manner within this thesis. Therefore, the scope of the thesis is limited as follows:

- Only river floods of low land rivers are considered.
- Flood consequence modelling is limited to direct tangible consequences (as introduced later) to residential buildings.

#### ***1.4 Outline of the thesis***

The thesis is structured in eight chapters. Chapter 1 (the present introduction) provides an overview of the topic, as well as objectives and scope of the thesis. Chapter 2 and Chapter 3 introduce the fundamental background knowledge by providing a relevant literature overview. Chapter 2 introduces the topics of engineering risk assessment, risk management and risk-based decision analysis. Chapter 3 examines the concepts introduced in Chapter 2, with a focus on floods. Chapter 4 defines and characterises hierarchical flood protection systems; their advantages are described and a modelling methodology is proposed for flood risk analysis, as well as a decision analysis, in the presence of hierarchical flood protection systems. Chapter 5 describes the developed probabilistic disaggregation model, which allows for disaggregating spatially aggregated portfolios of assets. This chapter is a paper submitted to the journal *Georisk* on March 12th, 2015. Chapter 6 describes the developed vulnerability model for residential buildings in floods; this chapter consists of a paper accepted for publication by the journal *Natural Hazards*. Chapter 7 describes an application of a hierarchical flood protection system to a study area in Switzerland. In the example application, a flood protection system with three hierarchy levels is proposed and optimised. The example also illustrates the application of the vulnerability model presented in Chapter 6. Chapter 8 concludes the thesis by highlighting and discussing its key findings and limitations and provides an outline of further research.



## **2 Engineering risk assessment and decision analysis**

This chapter introduces concepts inherent to engineering risk assessment, risk management and risk-based decision analysis. The concepts are introduced in general manner, but with a societal prospective, since flood risk management is generally a societal issue. Illustrative examples are taken from natural hazard risk assessment and management.

## **2.1 Introduction**

From a societal point of view, the role of a civil engineer is to support society in constructing, maintaining and decommissioning its infrastructure in a sustainable manner. The sustainability aspect is of great importance, as society has long recognised that natural and financial resources are limited (Faber 2007a). To maintain, and possibly increase, quality of life for future generations, society must make optimal use of its resources, since whatever resource is allocated in suboptimal manner today will be missing in the future.

Societal resource allocation can be divided into two tasks. The first task is the allocation of resources to different societal activities: power production, sewage treatment, transportation (while this task is often a prerogative of politicians, there are good arguments for basing such decisions on scientific knowledge). The second task is the prerogative of engineers: it requires optimally utilising resources allocated to a societal activity for the design, construction, inspection, maintenance and decommission of infrastructure and engineered systems necessary to the societal activity. In this context, optimality generally means identifying the system configuration promising the best cost-benefit ratio, while, at the same time, conforming to societal safety requirements of individuals and environment. This is a challenging task, generally entailing several decision problems, from general performance definitions to the design of small structural details. The task is further complicated by significant uncertainties embedded in each decision, stemming from inherent variability in, and incomplete knowledge of, the world. To facilitate identifying optimal system configurations, engineers develop mathematical models of reality to help consider and predict the behaviour of engineered systems in the present and future. These models allow comparison of benefits and costs entailed in each considered system configuration and ultimately allow identifying the optimal system configuration. For such analysis to be meaningful, it must consider the whole lifetime of a system and all events that might impact the system during its lifetime, as well as their potential consequences. As discussed in the following sections, this is the scope of engineering risk assessment, and decision analyses based on it are risk-based decision analyses.

## 2.2 Risk

Risk has many definition across research fields, see e.g. Vlek (1996). In an engineering context, risk has a predominantly negative connotation and the definition of UNISDR (2009) seems appropriate: “*The combination of the probability of an event and its negative consequences*”. This definition is formalised by Ciurean and Schröter (2013) as:

$$\text{RISK} = \text{HAZARD} \times \text{VULNERABILITY},$$

where hazard is a potential event with negative consequences and vulnerability is the susceptibility of a system exposed to the event. In a further formalisation, JCSS (2008) defines the risk  $r_{HZ}$  associated with a hazard event  $HZ$  as the product between the probability  $p_{HZ}$  that the hazard event occurs<sup>4</sup>, and the consequences  $c_{HZ}$  associated with the event:

$$r_{HZ} = p_{HZ} \cdot c_{HZ} \quad (2.1)$$

The total risk  $r$  for a system is determined by considering all hazard events  $HZ_m, m = 1, 2, \dots, M$  that can potentially affect the system:

$$r_{HZ} = \sum_{m=1}^M p_{HZ_m} \cdot c_{HZ_m} \quad (2.2)$$

## 2.3 Risk management

Society and individuals try to maintain, and possibly improve, present and future quality of life. Hazard events can be detrimental to quality of life; thus, society engages in risk management to minimise the probability of occurrence and consequences of hazard events.

Coherent risk management within, and across, application fields is challenging. The variety of engineered systems and hazard events to be considered is remarkable, as is the uncertainty in the behaviour of engineered systems. As such, risk management should be based on a generic framework allowing dependable representation of relevant

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<sup>4</sup> The probability of occurrence of a hazard event always relates to a time frame, which is generally chosen as 1 year in natural hazard risk assessment.

system characteristics, hazard events and consequences, under due consideration of uncertainties (JCSS 2008).

A general framework for risk management is illustrated in Figure 2.1. Risk management includes several activities, which, at the lowest level of analysis, can be seen as a continuous cyclical process of context and criteria definition, risk assessment, risk treatment and monitoring activities.

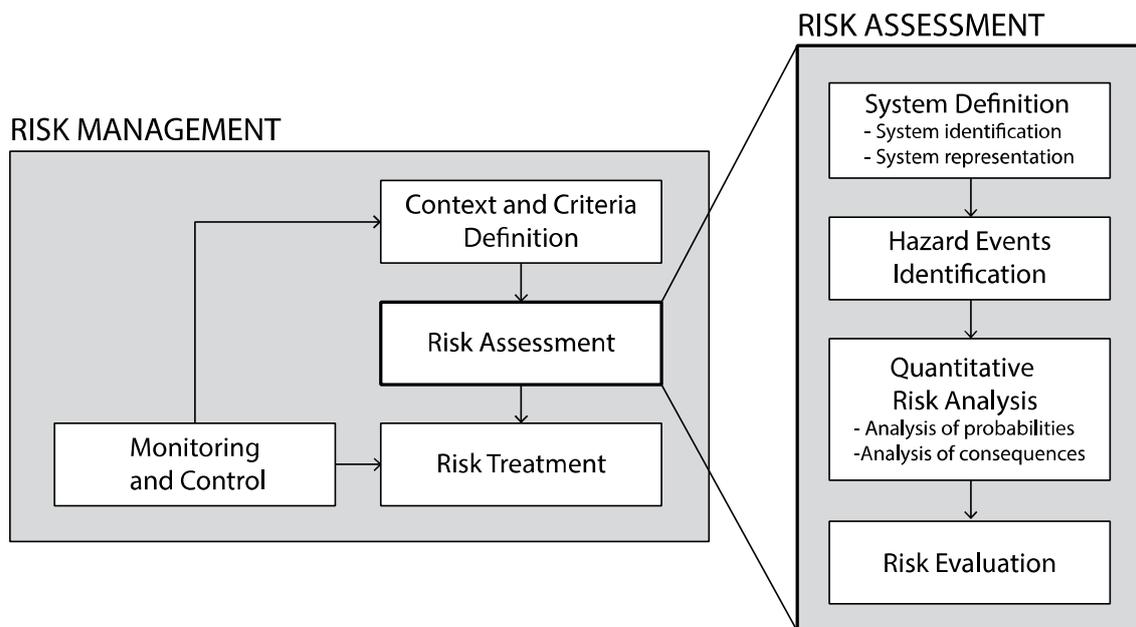


Figure 2.1 - Risk management framework, adapted from Faber and Stewart (2003) and Jonkman (2007)

Context and criteria definition provide the boundary conditions of risk management. For risk management to be meaningful, political, legal, social and financial boundaries of the analysed system need to be understood. Risk acceptance criteria of the society for persons and the environment also must be taken into account (Stewart and Melchers 1997). Failure to grasp boundary conditions may lead to insufficient, impractical or unworkable solutions, defying the very purpose of risk management.

The scope of risk assessment includes system identification and representation, identification of all potential hazard events and assessment of occurrence probability and consequences of each, i.e. actual quantification of risk. The steps entailed in risk assessment are illustrated in Figure 2.1 and thoroughly detailed in the next section.

Risk evaluation entails comparison of risk analysis results with the previously identified risk acceptance criteria, as well as evaluation of possible risk treatment measures to select an appropriate risk treatment strategy (thus, risk evaluation entails risk-based decision analysis, detailed in Section 2.5).

Risk treatment measures include all activities that impact risk or risk ownership; they may be classified as (see e.g. UNISDR 2009):

- risk avoidance measures,
- risk reduction measures and
- risk transfer measures.

Risk avoidance implies complete elimination of the possibility that certain hazard events occur. In Equation (2.2), risk avoidance corresponds to a reduction of number  $M$  of possible hazard events.

Risk reduction can be achieved in two ways: reducing the occurrence probability of hazard events and/or reducing their consequences, i.e. reducing either (or both) the multiplicands in Equation (2.2). Risk-reducing measures in natural hazard risk management can be divided into structural and non-structural measures. Structural measures entail physical modifications of the system: construction of a dike to reduce flood risk, construction of dampers in high rise buildings to reduce seismic risk, or implementation of roof ties to reduce strong-wind risk. Non-structural measures include measures aimed at a better functioning of a system before, during and after a hazard event. They include, amongst others, early warning systems, population education and risk-aware spatial planning. As mentioned in Chapter 1, in modern risk management, risk may be managed with a portfolio of integrated structural and non-structural measures (Jha *et al.* 2012).

Risk transfer does not change risk for the system; instead, financial implications of risk are transferred between two entities (persons or companies). An example of a risk transfer transaction is an insurance contract, where a person or company subject to risk, i.e. the insured, transfers the financial implication of a risk to an insurance company in exchange for a premium.

The last activity in the risk management cycle is continuous control and risk monitoring of the system. A system subject to risk is a system subject to change, and it is crucial to quickly identify system changes that may indicate a change in risk. As risk is abstract and often unapparent, monitoring focuses on the observation and measurement of risk indicators, defined by JCSS (2008) as “...*any observable or measurable characteristic of the system ... containing information about the risk*”. The results of risk monitoring and control activities may be utilised for a more accurate risk assessment and to measure the effectiveness of implemented risk treatment measures.

Several sources, e.g. Bayraktarli *et al.* (2004), distinguish three phases of risk management relative to the occurrence of a hazard event. These are:

- **Pre-event** – Before a hazard event occurs, risk management activities focus on the reduction of risk with optimal allocation of resources.
- **Event management** – During a hazard event, risk management activities are aimed at reducing and controlling consequences by providing help and rescue.
- **Post-event** – After a hazard event occurs, risk management activities include rebuilding and strengthening of affected systems, rehabilitation of infrastructure functionality and collection of information to improve future risk assessment.

These distinctions are important, since in each phase boundary conditions, available resources and risk treatment measures are different (Faber 2007b). However, in each phase, the framework illustrated in Figure 2.1 remains valid and should be applied.

#### **2.4 Risk assessment**

Risk assessment provides the quantitative basis for risk management. Several frameworks for engineering risk assessment are available in literature, e.g. Stewart and Melchers (1997) and JCSS (2008); the present section is primarily based on the latter. The considered framework for risk assessment is illustrated in Figure 2.1.

The first fundamental risk assessment step is the definition of the considered system; spatial and temporal scope of analysis is defined (system identification) and an appropriate representation of system characteristics, components and functionalities is ascertained (system representation). The next step is the identification of hazard events that could potentially affect the system. Given a list of all possible hazard events, a quantitative risk analysis is performed. Figure 2.2 illustrates a framework for risk analysis adapted from JCSS (2008), including three distinct modelling steps: hazard assessment, vulnerability assessment and robustness assessment. Hazard assessment entails the characterisation of each considered hazard event in terms of probability of occurrence and intensity. Vulnerability assessment is the characterisation of direct consequences (i.e. consequences for the system) from a hazard event. Robustness assessment is a characterisation of indirect consequences (i.e. consequences for everything outside the system due to system functionality loss) from the hazard event and/or direct consequences.

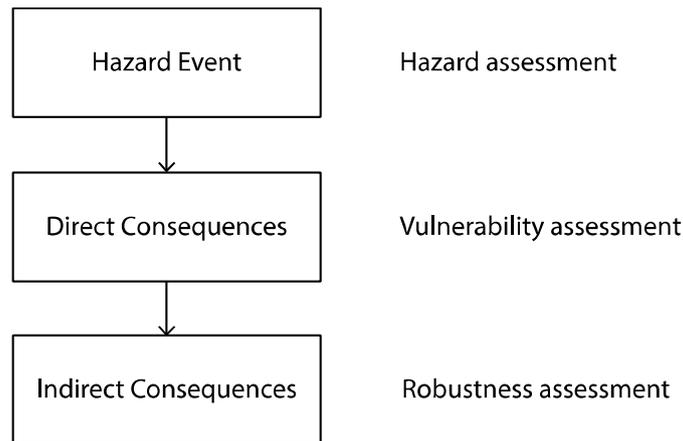


Figure 2.2 - Framework for quantitative risk analysis (adapted from JCSS 2008)

In the following sections, each step of the outlined risk assessment framework is detailed and the treatment of uncertainty in risk assessment is addressed, as it is of fundamental importance.

#### 2.4.1 System definition

Systems considered in an engineering risk assessment can range from a small component in an engineered structure to something as broad as the infrastructure network of a whole country. Temporal scope can also vary greatly; it is, however, common to consider the whole lifetime of a system, which may extend over several generations. With such an extensive range of possible spatial and temporal scopes, it is important to carefully define and delimit what is considered in a risk assessment. System identification involves the definition of a spatial and temporal scope of the system. It is as much about exclusion as it is about identification: all aspects not relevant to risk assessment must be excluded. Once the system is identified, it has to be appropriately represented. System representation concerns the translation of the actual system into an abstracted logical and/or mathematical model. Generally, many different system representations are possible, each with different level of detail and sophistication. In the choice of an appropriate system representation, compromise is often necessary between level of detail in the system representation and computational demands that follow from the chosen representation. In general, a system representation should be detailed enough to capture relevant system characteristics, hazard processes and consequences, while still allowing for reasonable computational times. In practice, the level of detail of system representation may also be dictated by the amount of available information. Small and simple systems, e.g. one engineered structure, can

generally be quite precisely understood, allowing for a detailed system representation. Conversely, when analysing large and complex systems, such as an infrastructure network or a portfolio of structures, available knowledge is often less detailed, and the system representation more coarse.

Complex systems may be represented as a collection of system components and functional interrelations between components. While the overall system complexity can be prohibitive to model, behaviour of individual components is often reasonably well understood and thus easier to model. In this context, it is important to notice that every component can be a system in itself. For instance, as illustrated in JCSS (2008), a system could be a road network, and a bridge in the network a system component; in turn, a bridge could be the considered system with the bridge deck, columns and cables its components.

Whichever system representation and level of detail is chosen, it has to represent available knowledge in a consistent manner, and should be easy to update when new knowledge becomes available. It is also important to consider and represent uncertainty: engineers have limited and incomplete understanding of reality, which has to be reflected in the system representation.

### 2.4.2 *Uncertainty*

Systems and processes considered in a risk assessment are inherently uncertain. For instance, time of occurrence and intensity of a natural hazard event, or development of a fatigue crack can generally not be known with precision. Furthermore, when dealing with complex systems and time horizons stretching over decades, knowledge about the future state of the system is always imperfect and therefore subject to uncertainty. For a meaningful risk assessment it is important to capture and represent uncertainty in an appropriate manner, in the words of Faber (2005): “ ... *only when the origin and the nature of the prevailing uncertainties are fully understood, can a consistent probabilistic modelling be established, allowing for rational decision making regarding risk reduction*”.

Although uncertainties can be classified in several ways, in engineering risk assessment it is common to differentiate uncertainties according to their type and origin, as natural variability in the system, model uncertainties and statistical uncertainties (see e.g. Faber 2005). Natural variability is often referred to as aleatory uncertainty; model and statistical uncertainties as epistemic uncertainties. Aleatory uncertainty is

considered inherent to nature and cannot be reduced with mere data collection (Ditlevsen and Madsen 2007). Conversely, epistemic uncertainty can be reduced when new data on the system becomes available. In practice, the distinction between aleatory and epistemic uncertainties is often not trivial: it depends on the chosen system representation (Kiureghian and Ditlevsen 2009) and might change with time (Faber 2005). Nevertheless, the proposed distinction is useful to frame the sources of uncertainty and distinguish their treatment in risk assessment.

According to Paté-Cornell (1996), uncertainty may generally be considered in three different ways in natural hazard risk assessment:

- **Assessment of aleatory uncertainties only** – In this approach, best estimates of parameters and models are utilised and epistemic uncertainty is disregarded.
- **Integrated treatment of aleatory and epistemic uncertainties** – Here, epistemic uncertainty is integrated with aleatory uncertainty. Through the integration, the impact of epistemic uncertainty on risk is not explicit in the result.
- **Separated assessment of aleatory and epistemic uncertainties** – Here too, epistemic uncertainty is considered, but in an explicit manner. In a general approach, risk assessment may be carried out conditionally on epistemic uncertainty parameters. In the result, epistemic uncertainty is explicit and risk may be illustrated with (epistemic) uncertainty bounds.

Integrated treatment of uncertainties can be approached within the Bayesian probability framework, which allows for coherently integrating knowledge from different sources and updating probabilities when further knowledge becomes available (i.e. reducing epistemic uncertainty). Several authors recommend (Paté-Cornell 1996, Faber 2007b) or apply (Bayraktarli *et al.* 2005, Straub 2005, Nishijima and Faber 2007) the Bayesian probability framework for engineering risk assessment.

Hall and Solomatine (2008) acknowledge the value of integrated uncertainty treatment with the Bayesian probability framework. However, they highlight a communication problem its usage may entail; the public may be surprised by a reduction in risk estimate due to mere data collection, which may, in turn, lead to a loss of trust in the risk assessment and authorities. The same paper highlights an advantage of separated uncertainty treatment: it allows for calculating the probability that an apparently optimal decision alternative may turn out to be sub-optimal.

Choosing a system representation and modelling approach often involves a choice between simple models with few explanatory variables, but large aleatory and epistemic (model) uncertainty, and more complex models, with many explanatory variables, and

consequently large epistemic (statistical) uncertainty. Both approaches can be valid, but in models dominated by statistical uncertainties, potential for future uncertainty reduction through data is larger, and thus such a model is generally preferable (Faber 2005).

#### 2.4.3 Hazard assessment

Prior to hazard assessment, relevant hazard processes must be identified, that is, anthropogenic, physical, chemical and biological processes, which may be the source of hazard events, are identified. Hazard processes may include exogenous hazard processes (i.e. caused by a process or agent external to the system: for example, a natural hazard event) and endogenous hazard process (i.e. caused by a failure within the system without a focused impact from outside: for example, a fatigue failure), see e.g. Stewart and Melchers (1997). With relevant hazard processes identified, the hazard assessment comprises two parts: first, the characterisation of hazard processes to determine the probability and magnitude of hazard events arising from it and second, characterisation of the hazard action (in terms of energy, acceleration, pressure, etc.) impacting the system, given a hazard event of a certain magnitude. The two parts are described in more detail in the following, using examples from natural hazard modelling.

The first part characterises the probability of hazard event occurrence with a specific magnitude in time and space. It entails hazard source characterisation based on historical data, risk indicators and/or expert knowledge. For instance, the hazard source of earthquakes is generally a tectonic fault and, for a given fault, probability of occurrence and magnitude of the earthquake are related by a functional relationship first described by Gutenberg and Richter (1942); this relationship can be parameterised from historical data. For river floods, the hazard source is generally a hydro-meteorological process leading to extreme rainfall in the river catchment. While it is possible to model the meteorological process and the extreme rainfall event, it is just as common to avoid modelling these complex processes and directly model flooding from the river discharge process. To characterise river discharge probability distribution, long series of historical discharge data are often available in developed countries; these allow characterising the probability of occurrence of extreme discharge events through extreme value theory. Modelling of hazard source process from historical data requires particular attention, as

stationarity, or the presence of trends, needs to be assessed. Given climate change, the stationarity of hydro-meteorological processes is doubtful.

The second part of a hazard assessment is the characterisation of a hazard event development, in terms of energy transfers from the hazard source to all locations in the considered system, in order to characterising hazard actions throughout the system. Hazard actions are generally described with hazard indices, i.e. physically observable, or easily derivable, quantities that characterise the energy impacting the system in consequence of a hazard event. The physical process that transfers energy from the hazard source to a system location is obviously a function of the type of hazard in question, as exemplified in the following.

For earthquakes, seismic waves spread from the earthquake hypocentre through rocks and terrain to the earth surface, where they may be felt as a ground motion; the ground motion intensity is generally a function of distance to the hypocentre and geological characteristics. Intensity of the ground motion for a given hazard event, can be characterised with different hazard indices like peak ground acceleration or spectral acceleration, which may be modelled with attenuation functions (see e.g. Ambraseys and Bommer 1991).

In a river flood, excess river discharge leaves the river bed and inundates surrounding dry land. The flood development is determined by surrounding terrain and surface friction and is driven by gravitational forces. Hazard indices that might be used to characterise hazard actions on the system are water depth, water flow velocity and water contamination (see Kelman and Spence 2004 for a comprehensive list). The flood development process is generally modelled with flow routing models, allowing for determining floodplain water discharge.

When selecting which hazard index/indices to consider in a risk assessment, it is important to consider that hazard indices form the interface between hazard assessment and vulnerability assessment; the chosen hazard indices should thus correlate with consequences.

#### 2.4.4 Consequence assessment

Consequences of a hazard event include all changes to the system induced by the event. In engineering risk assessment, consequences generally have a negative connotation, i.e. they entail a deterioration of the system, leading to a loss of value, lives and/or functionality. The nature of consequences depends on, and can vary greatly between, the system(s). Therefore, no general approach for the assessment of consequences can be provided. Instead, a common categorisation of consequences is described alongside several modelling principles common in engineering risk assessment.

It is useful to distinguish different types of consequences. A first distinction is commonly made between direct and indirect consequences, see e.g. JCSS (2008) and Merz, Kreibich *et al.* (2010). JCSS (2008) describes direct consequences as “... *consequences associated with damages or failures of the constituents of the system.*” Indirect consequences are induced by direct consequences, and are described by the same source as “...*any consequences associated with the loss of functionalities of the system and by any specific characteristic of the joint state of the constituents and the direct consequences themselves.*”

Several sources, e.g. Cochrane (2004), note that the distinction between direct and indirect consequences is not absolute and depends on the system definition, as exemplified in the following. Consider that a bridge within a transportation network is damaged by a hazard event. When the bridge is the system of interest, then the bridge closure is a direct consequence and the transportation network distribution is an indirect consequence. On the other hand, when the transportation network is the system of interest, the direct consequence of the bridge damage might be network disruption and the indirect consequence might be much broader and include business interruption for companies utilising the transportation network (JCSS 2008).

Consequences can be further classified as tangible and intangible consequences; the former can easily be quantified in monetary terms, the latter cannot (Kübler 2007). Examples of consequences and a possible classification are given in

DIRECT		INDIRECT	
TANGIBLE	INTANGIBLE	TANGIBLE	INTANGIBLE
<ul style="list-style-type: none"> <li>• Damage to private buildings and contents</li> <li>• Damage to infrastructure, e.g. roads, railways, agricultural soil, destruction of harvest</li> <li>• Damage to livestock</li> <li>• Evacuation and rescue measures</li> <li>• Business interruption inside the affected area</li> <li>• Clean up costs</li> </ul>	<ul style="list-style-type: none"> <li>• Loss of life</li> <li>• Injuries</li> <li>• Loss of memorabilia</li> <li>• Psychological distress</li> <li>• Damage to cultural heritage</li> <li>• Negative effects on the ecosystem</li> </ul>	<ul style="list-style-type: none"> <li>• Disruption of public service outside the affected area</li> <li>• Induced production losses to companies outside the affected area</li> <li>• Cost of traffic disruption</li> <li>• Loss of tax revenue due to migration of companies in the aftermath of an event</li> </ul>	<ul style="list-style-type: none"> <li>• Trauma</li> <li>• Loss of trust in authorities</li> </ul>

Figure 2.3 - Categorisation of consequences according to Merz, Kreibich *et al.* (2010)

As illustrated in Figure 2.2, direct consequences can be assessed with a vulnerability model, indirect consequences with a robustness model. Most available vulnerability and robustness models only characterise tangible consequences. In the following, two vulnerability modelling approaches are described; then, observations are made on modelling of robustness, before the section is concluded with remarks on intangible consequences modelling.

Two common vulnerability modelling approaches are the damage function approach and fragility approach. As the name suggests, in the damage function approach, consequences are modelled in function of one or more hazard indices (see Figure 2.4, left). Damage functions can either be relative (expressing consequences as a percentage of the value of the underlying asset) or absolute (consequences are directly expressed in monetary terms), and they are often derived from statistical analysis of empirical damage data from past hazard events, see e.g. the flood vulnerability curves in Maiwald (2007). Alternatively, damage functions can be determined through expert knowledge with what-if analyses and engineering models on hypothetical systems, see e.g. the flood vulnerability curves in Penning-Rowsell *et al.* (2010).

In the second type of vulnerability modelling, the fragility approach, consequences are characterised in two modelling steps. In the first step, a fragility model describes the probability of the system being in a physical damage state  $DS_i$  (physical damages may be defined in descriptive terms, e.g. “No damage”, “Cracking in walls and broken

windows”, “Partial wall failure”, “Total collapse”) for given hazard index (see Figure 2.4, right). In the second step, the direct consequences  $D$  are quantified in function of the physical damage state of the system,  $D = f(DS_i)$ . Similar to damage functions, fragility models can be statistically inferred from empirical damage data. However, when considering engineered systems, it is common to characterise the probability of the system being in each damage state with an engineering model. Example applications of the fragility approach are found in earthquake risk assessment (Bayraktarli *et al.* 2005), flood risk assessment (De Risi *et al.* 2013), strong wind risk assessment (Rosowsky and Ellingwood 2002) and multiple hazards risk assessment (Lee and Rosowsky 2006).

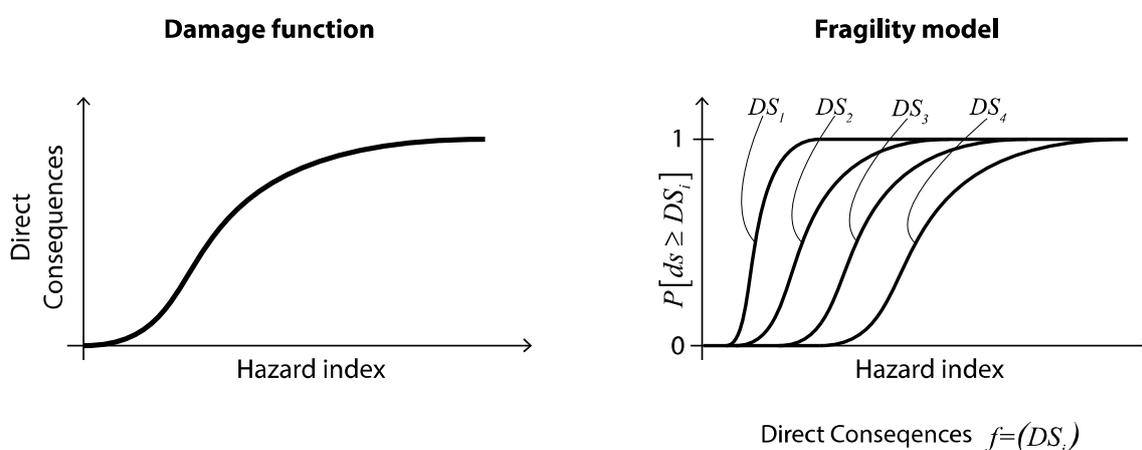


Figure 2.4 - Schematic illustration of different types of vulnerability models: on the left a damage function, on the right a fragility model

Robustness models characterise indirect consequences, which can be modelled either in function of hazard indices, direct consequences or both. The assessment is more complex than that of direct consequences, since the nature of indirect damages is even more diverse (cf. Figure 2.3). Moreover, the functional relationships between systems or components which lead from direct to indirect consequences are often complex and require a detailed analysis. As such, no comprehensive list of modelling approaches can be provided here. Instead, a general approach to measure the robustness of a system, which may be used to quantify indirect consequences, is detailed in the following.

The term robustness may be associated with many different system properties: for a comprehensive list, see Baker *et al.* (2008). Generally, systems are considered robust if the consequences of a system failure are not disproportional to the effect causing the failure. As such, robustness is a desirable property of systems, and can be considered in

decision analysis. A robustness index is proposed in JCSS (2008), which relates direct risk to total risk (i.e. the sum of direct and indirect risk). Whereas JCSS (2008) specifies this robustness index for structural systems, it can just as well be used for other types of systems, e.g. social systems. When direct consequences are known, the robustness index allows for simple modelling of indirect consequences.

Finally, the modelling of intangible consequences is addressed. Approaches are available to quantify intangible consequences in non-monetary terms: e.g., number of fatalities, number of injuries, or pollutant concentration. For instance, number of fatalities and injuries are modelled in Jonkman (2007) for floods and in Coburn and Spence (2002) for earthquakes. In principle, this type of quantification is sufficient and can be considered in a decision analysis with a multi-criteria utility function. However, full consistency in decision alternative evaluation may only be achieved when all consequences are expressed in terms of a common utility, e.g. in monetary terms. Although it is controversial and difficult to translate these consequences into monetary terms (Ayres *et al.* 1998), several methodologies are available to assign a monetary value to intangible consequences, e.g. stated preferences, revealed preferences, or informed preferences methodologies (see e.g. Schubert 2009). In engineering decision analysis, the Life-Quality Index (LQI) methodology (Nathwani *et al.* 1997) is gaining increasing attention. LQI is a compound indicator of human welfare, which can be utilised to evaluate the effectiveness of societal risk management activities.

Once a monetary value is assigned to intangible consequences, decision analysis reduces to a simple comparison of costs and benefits of decision alternatives in monetary terms (Faber 2005).

## **2.5 Engineering decision analysis**

Design of structures, evaluation of existing structures and optimisation of maintenance are examples of decision problems which can be approached with engineering decision analysis (Kübler 2007). Engineering decision analysis is the comparison of a set of possible system configurations (i.e. decision alternatives), with the goal of identifying the optimal one. To compare decision alternatives in an objective manner, a utility function is formulated allowing measurement of costs and benefits of each decision alternative over the considered time. In the general approach, the optimal decision alternative provides largest expected utility, while conforming to societal risk acceptance criteria. Engineering decision analysis is part of the normative branch of

decision theory, which aims at identifying and prescribing an optimal decision alternative according to rational criteria.

While certain subjectivity is granted in decision problem formulation, the decision maker bears the ultimate responsibility for a decision and is always subject to societal laws and preferences, particularly in respect to safety of the individual and the environment; decisions must conform to societal risk acceptance criteria (Stewart and Melchers 1997).

### 2.5.1 Decision alternatives and utility function

An engineering decision analysis requires the definition of a set of decision alternatives  $\mathbf{A} = \{a_1, a_2, \dots, a_N\}$  available to the decision maker, where  $a_i, i = 1, 2, \dots, N$  are decision alternatives. The nature of decision alternatives depends on the type of decision problem. For instance, in the conceptual design of a bridge, two decision alternatives may be given by a suspension bridge and a cable-stayed bridge. Conversely, in the detailed design of a bridge, decision alternatives may pertain to different dispositions of reinforcement bars in the bridge deck. Furthermore, in a decision problem involving inspection planning for an existing structure, decision alternatives may involve different inspection methods and time intervals between inspections.

In an engineering decision analysis, decision alternatives are ranked according to a utility function, to identify the decision alternative with the largest expected utility over the given time span. Von Neumann and Morgenstern (1943) lay the theoretical foundation for utility function formation. The utility function  $U(a_i, \Psi(a_i, \Theta))$  for decision alternative  $a_i$  normally considers all costs and benefits entailed by the decision alternative in the form of consequences  $\Psi(a_i, \Theta)$ . The consequences are considered to be a function of future system characteristics, which are uncertain, and are thus represented by random variable  $\Theta$  (Benjamin and Cornell 1970).

The utility function  $U(a_i, \Psi(a_i, \Theta))$  often considers costs and benefits incurred at different times over the lifetime of a system, which cannot be directly summed due to time value of money. To allow a comparison of decision alternatives from a current perspective, the utility function should consider the discounted net present value (NPV) of future costs and benefits. Whereas the mathematics of discounting are well established, see e.g. Mankiw (2011), opinions diverge on the choice of an appropriate discount rate, which should include considerations on time preference, economic growth and sustainability (see e.g. Nishijima *et al.* 2006). According to HM Treasury

(2003), a valid range for engineering projects with a long life time, in developed countries, is 2.5-3.5% . In France a rate of 4% is utilized.

### 2.5.2 Decision problems

Three types of decision problems are defined , depending on knowledge of the future state of the system  $\Theta$  (Luce and Raiffa 1957):

- **Decision making under certainty** – A decision problem is considered “under certainty” when the state of the system - after each decision - is known. Therefore, consequences  $\Psi(a_i)$  are only a function of the decision alternative and the optimal decision is simply identified by choosing the alternative with highest utility value.
- **Decision making under risk** – Decision making under risk occurs when the future state of the system  $\Theta$  is not known, however its probabilistic characterisation  $f_{\Theta}(\theta)$  is known. Assuming that the decision maker is risk neutral, the optimal decision  $a^*$  is given by the alternative with the largest expected utility over all realisations of  $\Theta$  , i.e.

$$a^* = \max_a E_{\Theta} [U(a, C(a, \Theta))] \quad (2.3)$$

- **Decision making under uncertainty** - Here, future states of the system  $\Theta$  , as well as its probabilistic characterisation  $f_{\Theta}(\theta)$  , are not known. Under these circumstances, the optimal decision alternative cannot be uniquely identified and several decision making criteria are available, e.g. maximin utility, Niehans-Savage criterion, Hurwicz  $\alpha$ , and Laplace’s principle of insufficient reason. Each criterion defines a rule to select a decision alternative in accordance with the preferences of the decision maker (see e.g. Kübler 2007 for details).

Engineering decision problems are generally classified as either “under risk” or “under uncertainty”. However, for practical purposes, most cases of decisions “under uncertainty” can be translated to a decision “under risk” by postulating  $f_{\Theta}(\theta)$  based on experience, data, or personal preference (Kübler 2007). When postulating  $f_{\Theta}(\theta)$  , the question arises whether further data and knowledge on  $\Theta$  should be obtained before a decision is made. The Bayesian decision theory (Raiffa and Schlaifer 1961, Benjamin and Cornell 1970) provides a formal basis for answering this question; the theory is widely utilised in engineering decision making). In Bayesian decision theory, the distinction between prior, posterior and pre-posterior decision analysis is made:

- **Prior decision analysis** – In the simplest form of decision analysis, expected utility for each alternative is evaluated with statistical information and probabilistic modelling available at the time of the decision.

- **Posterior decision analysis** – Posterior decision is structured like a prior decision analysis, however expected utilities are calculated based on probabilities updated to reflect changes in the system or increased knowledge on the system (i.e. from data collection). Posterior risk analysis is often used to analyse the impact of risk reducing measures.
- **Pre-posterior analysis** – The pre-posterior decision analysis allows addressing questions on whether further data on the system should be collected before making the actual engineering decision. The practice involves a posterior analysis for each possible data collection outcome, assuming that data collection outcome follows the prior probability distribution and considering uncertainties associated with observation and interpretation of the outcome. The pre-posterior analysis may be interpreted as a posterior decision analysis made before new information is actually collected.

### 2.5.3 Risk acceptance

Cost-benefit optimality is always desirable; however, society has further requirements for engineered systems. An acceptable decision alternative must comply with societal standards in terms of personal safety (injuries and fatalities) and environmental consequences (e.g. pollution, loss of biodiversity), see JCSS (2008). These standards are referred to as risk acceptance criteria and are available in different forms.

The ALARP criterion (“As Low As Reasonably Possible”) was developed by US Nuclear Regulatory Commission and is widely used in practice (Stewart and Melchers 1997). A system meets the ALARP criterion if it can be demonstrated that costs for further risk reduction are disproportionate compared to its potential benefits. ALARP is its non-committing terminology; “reasonably” and “possible” leave room for interpretation.

A second type of risk acceptance criteria are safety goals, which specify a minimal safety requirement e.g. in terms of annual fatalities related to an activity (Stewart 2010). Safety goals are sometimes given in the form of F-N curves, relating frequency to magnitude of consequences, and allowing differentiation between acceptance criteria for societal risks and individual risks, see Faber and Stewart (2003).

In recent decades, risk acceptance criteria based on the “societal willingness to pay” have emerged. They determine how much society is willing to invest in a marginal increase of safety for its members, which directly implies the risk the society is willing to accept. This approach may be based on the LQI methodology, see Nathwani *et al.* (1997) and Rackwitz *et al.* (2005).

## **3 Flood risk management**

In this chapter, engineering risk assessment and decision analysis topics are revisited, in the context of flood risk; particular attention is paid to river floods and direct consequences for residential buildings, the focus of this thesis.

### 3.1 Flood

Flood can be defined as “*the overflow of a large amount of water beyond its normal limits, especially over what is normally dry land*” (OED 2014). This might well be the only common element for all floods, which otherwise vary greatly in source, cause and characteristics.

Floods generally originate from overflowing water bodies or streams, e.g. a river, lake or sea. Heavy rainfall can be the direct source of floods when rainwater cannot run off fast enough, due to saturated soils or insufficient capacity of the urban drainage system. Flood causes include long-lasting rainfall, snowmelt, dam break and temporary sea-level rise due to low pressure weather systems (IPCC 2014b). Next to source and cause, a flood event can be characterised with a plethora of attributes, e.g. description of its development in time and space in terms of water depth, water flow velocity, etc.

A categorisation of different flood types and sources is given in Jonkman (2005) and Swiss Re (2007), the bases for the following summary:

- **River floods** – River floods occur when a water stream carries excessive water, which may leave the river bed. They generally occur due to heavy rainfall in the river catchment, sometimes in combination with other causes, e.g. snow melt, blockage of the river bed by ice jams, landslides or a breach in a flood protection structure. River floods may be categorised as “slow rising” or “flash flood”. The former occur in the lower course of rivers and are characterised by slow water rise and slow water flow velocity. Slow rising floods can usually be predicted several hours or days in advance, thus allowing emergency risk management measures to be put in place, e.g. evacuations and local flood barriers. In contrast, flash floods occur in mountainous regions and are characterised by very quick water rise, high water flow velocity and debris flow. They are generally the consequence of extreme rainfall, but can also have other causes, e.g. landslides into a water basin, see Kilburn and Petley (2003). The warning time before a flash flood is often very short.
- **Coastal floods** – Coastal floods occur on the shores of lakes and seas and have two main causes. Lakes can overflow due to heavy rainfall in the catchment of the lake and/or following the decision by authorities to limit the outflow of a lake to protect lowlands from river floods. The second main cause of coastal floods is an atmospheric low-pressure system over sea, which may cause the sea level to temporarily increase by several meters (storm surge).
- **Areal floods** – Areal floods occur in flat lowlands when rainwater from extreme rainfall cannot run off fast enough and thus accumulates on the surface. In urban environments, they occur when storm sewer capacity is insufficient to drain rainwater fast enough; in rural areas, when the soil is saturated.

Source and type of flood are relevant to flood risk management for at least two reasons. First, they are factors in the hazard assessment; second, the effectiveness of individual risk management measures may vary depending on the flood type.

### 3.2 Flood risk

Following the definition of risk in Section 2.2, flood risk is characterised by the probability of flooding and its consequences.

Let  $\mathbf{X}$  be a random vector characterising flood hazard<sup>5</sup> for the system of interest and  $C$  the flood consequences for the system. Furthermore, let  $f_{\mathbf{x},c}(\mathbf{x},c)$  be the joint probability density function of  $\mathbf{X}$  and  $C$ . Then, flood risk  $r$  can be characterised in general manner as:

$$r = \int_{D_c} \int_{D_x} c \cdot f_{\mathbf{x},c}(\mathbf{x},c) d\mathbf{x}dc \quad (3.1)$$

where  $D_c$  and  $D_x$  are the domains of  $C$  and  $\mathbf{X}$  respectively.

### 3.3 Flood risk management

Flood risk management is defined in Schanze (2006) as the “*continuous and holistic societal analysis, assessment and mitigation of flood risk*”. Recognising the importance of a holistic and integrated approach to flood risk management is relatively recent (ICE 2001, Hall 2011). Previously, efforts focused on flood protection, rather than flood risk management. That is, protection structures were designed with one particular design event in mind (e.g. 200 year event) and little consideration for system vulnerability, system robustness and residual risk. The new paradigm of flood risk management has developed over the past decade and can be summarised in three points (Merz, Hall *et al.* 2010):

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<sup>5</sup> As described in Section 2.4.3 hazard assessment may entail the characterisation of hazard sources, as well as the characterisation of hazard actions throughout the system. In the present and following chapters the designation “hazard” refers to hazard actions (e.g. water depth, water flow velocity, etc.). Conversely, “hazard source” refers to the flood source (e.g. river discharge).

- **Risk based approach** – Consideration is given to all possible flood events and their consequences.
- **Formal decision analysis** – Flood risk management decisions are made on the basis of flood risk assessment and cost-benefit analysis.
- **Integrated systems approach** – Flood risk management strategies are developed, including structural and non-structural risk management measures.

Approaches to, and frameworks for, flood risk management are described in e.g. ICE (2001), Schanze (2006), Merz, Hall *et al.* (2010), Hall (2011) and Jha *et al.* (2012). They are generally in good agreement with the risk management framework outlined in Section 2.3. Several aspects of interest are highlighted in the following.

Merz, Hall *et al.* (2010) emphasises that, for flood risk management to be effective and sustainable, it must consider uncertainty and non-stationarity of flood risk. The authors underline the importance of optimising decisions for cost-benefit optimality, as well as for robustness<sup>6</sup> and flexibility.

Jha *et al.* (2012) emphasises the concept of integrated flood risk management, defined as “...a combination of flood risk management measures which, taken as a whole, can successfully reduce urban flood risk.” The concept is based on the recognition that large flood protection structures alone cannot effectively manage flood risk (because of large upfront costs, long planning and construction times and changing flood risk). The report proposes managing residual risk of large protection structures by implementing complementary structural and non-structural measures. The value of non-structural measures, including early-warning systems and population education, is emphasised; these steps can have a large impact on preparedness and loss reduction, without requiring large upfront investments.

The following sections examine flood risk assessment (next section) and flood risk management measures (Section 3.5).

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<sup>6</sup>The meaning of the term "robustness" is different depending on whether it refers to a system or a decision. A robust system is described in Section 2.4.4; a robust decision performs well under a large number of future states of the world, i.e. its performance is insensitive to uncertainty variables and parameters.

### 3.4 Flood risk assessment

Flood risk is assessed according to the general risk assessment framework illustrated in Figure 2.1. It includes system definition, hazard identification, risk analysis and risk evaluation. The approaches to system definition, hazard identification and risk evaluation described in Chapter 2 are valid for flood risk assessment and no further details are given here. However, flood risk analysis involves unique aspects, explained in the following.

#### 3.4.1 Flood risk analysis

A common way to analyse flood risk is the Source-Pathway-Receptor framework (ICE 2001), illustrated in Figure 3.1. Sources of floods are described in Section 3.1. Pathways are discharge routes through which water propagates from the source to locations where people, buildings and other assets are located. Receptors are people, buildings and assets that could be potentially damaged by floods. In Schanze (2006) and Hall (2011), the framework is extended with a fourth element: “harm”, i.e. flood consequences.

Individual modelling elements in the Figure 3.1 are described in more detail in the next sections, with a focus on aspects relevant to this thesis, river floods and direct effects on residential buildings.

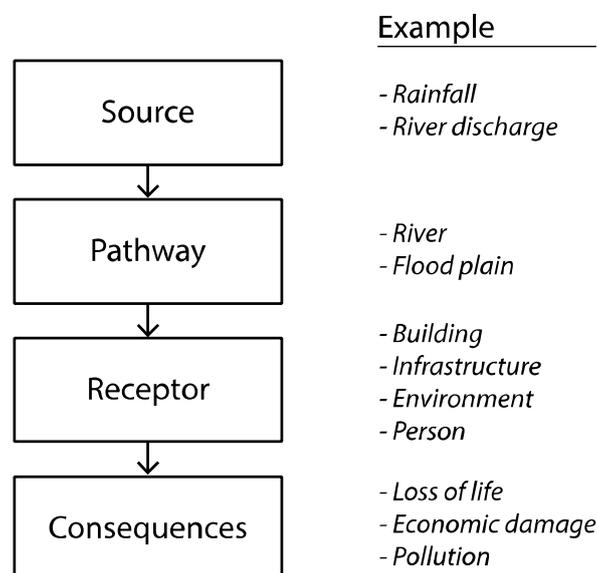


Figure 3.1 - Schematic representation of the Source-Pathway-Receptor flood risk assessment framework (adapted from Hall 2011)

### 3.4.1.1 Source modelling

Processes leading to heavy rainfall or excessive river discharge, which then may cause floods, are comprehensively described with the hydrologic cycle, see Chow *et al.* (1988). In broad strokes, the hydrological cycle includes rainfall, ground infiltration and surface runoff, river runoff, storage (in lakes and seas) and evaporation. As the hydrologic cycle is a continuous process, the step of the cycle, that can be considered the actual source of a flood event, is not unequivocally identifiable. However, it is reasonable to consider the meteorological processes that lead to heavy rainfall, or the river discharge process, as hazard source and therefore start flood risk analysis from there.

When river discharge is considered as hazard source, probabilistic characterisation of the river discharge process can be derived from discharge time series data (generally available for most rivers in developed countries) by employing extreme value theory. From it, the probability of flood event occurrence can be derived. This approach avoids the modelling of complex, highly uncertain and computationally intensive meteorological processes. However, an underlying assumption of this approach is the stationarity of river discharge process. As previously mentioned, this assumption is challenged by climate change research. When the impact of climate change on flood risk is of interest, it is advisable to consider the meteorological process as hazard source by using global or regional circulation models as inputs (see e.g. Feyen *et al.* 2012).

### 3.4.1.2 Pathway modelling

The “pathway” from source to receptor is modelled as water flow routing in the river bed and on the flood plain. Several modelling methodologies are available, which can be broadly categorised as lumped system flow routing and distributed system flow routing. Lumped flow routing models characterise the water flow as a function solely of time; conversely, distributed system flow routing models water flow as a function of both time and space (Chow *et al.* 1988).

Lumped flow routing models are based on the continuity equation of hydrology and functional relationships between discharges in different parts of the studied hydrological system. It is, generally less computationally intensive and less accurate than distributed flow routing. Nevertheless, it is used in applications where a large number of flow simulations is required, e.g. to estimate hazard uncertainty (Apel *et al.* 2006), or to consider a large number of dike breach scenarios (Gouldby *et al.* 2008, Bernini and Franchini 2013).

In distributed system flow routing models, water flow is most commonly modelled with an approximation of the Navier-Stokes equations for incompressible fluid motion. Most models rely on the St. Venant approximations, i.e. one-dimensional channel flow equations and two-dimensional shallow-water equations. See Chow *et al.* (1988) for equations, underlying assumptions and numerical solutions.

Several academic and commercial distributed flow routing models are available. One-dimensional river channel flow models include Mike 11 (DHI 1995) and HEC-RAS (USACE 1995). They offer a computationally efficient solution for river flow routing, require little input data; under certain conditions, they can also be used to model flood plain routing. However, they do not account well for complex terrain and water spreading in more than one direction. Two-dimensional routing models include TELEMAC-2D (Galland *et al.* 1991), SOBEK (Deltares 2014), ISIS2D (CH2MHill 2014) and Tuflow (Syme 2001). They allow detailed flow routing over a flood plain and can account for complex terrain. However, they require more computational resources and runtime. LISFLOOD-FP (Bates and De Roo 2000) couples a one-dimensional river channel routing model with a two-dimensional raster-based flood plain routing model; for both routing models, several numeric algorithms are available to describe water flow routing.

An overview and evaluation of these, and several other flow routing models is found in Wicks *et al.* (2004). Syme (2001) provides a performance comparison of ISIS2D, LISFLOOD-FP, TELMAC-2D and TUFLOW. Horritt and Bates (2002) compare HEC-RAS, LISFLOOD and TELEMAC-2D.

#### 3.4.1.3 Receptor modelling

A receptor is a person, building or other object subject to flood risk. In engineering risk assessment, these may be called system constituents; in an insurance context, they are referred to as exposure<sup>7</sup>.

Receptor modelling is based on system representation described in Section 2.4.1. It entails the characterisation of all objects at risk, i.e. their tangible and intangible valuation, location, functionality and other features relevant to consequence assessment, like mobility and flood susceptibility.

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<sup>7</sup> In the paper reproduced in Chapter 5 the term "exposure" is also utilised to identify a portfolio of receptors.

In this thesis, the focus is on building receptors. For buildings, relevant features include structural details which may influence vulnerability: construction material, door types and presence or absence of basements. Tangible valuation of buildings may vary depending on the purpose of the analysis, as replacement value or depreciated value may be considered (see e.g. Jongman *et al.* 2012). Intangible valuation is inherently more challenging; however, valuation methodologies are available, e.g. in Holicky and Sykora (2010), for historical heritage building valuation.

Receptors may be modelled with different levels of detail and spatial resolution, ranging from detailed modelling of individual receptors to aggregated modelling over whole administrative regions. The choice of detail level and spatial resolution are often related to study area size, with decreasing level of detail and resolution for increasingly larger study areas. At the largest scale of modelling, macro-scale exact locations and characteristics of receptors are not considered; the value of receptors is aggregated for whole administrative regions. On meso-scale, receptor characteristic may be considered in a statistically aggregated manner and the value of receptors is aggregated at grid cell level and/or by land use. On micro-scale, location, characteristics and valuation of receptors are considered in detail for individual receptors.

In addition to spatial resolution of choice, level of receptor modelling detail depends on the quantity and quality of available data. In developed countries, authorities generally have detailed data on location and main features of building and infrastructure receptors. However, due to data protection rules, such data might not be publicly available, or only available in statistically aggregated form.

### 3.4.1.4 Consequence modelling

Flood consequences are modelled with vulnerability and robustness models. Vulnerability models with different degrees of sophistication are discussed in literature for a wide variety of receptors; however, fewer robustness models seem available. References to literature models are given throughout this section.

Vulnerability models for residential building in floods aim at relating hazard indices and building characteristics to direct consequences. The hazard index with the strongest correlation to direct consequences is water depth, followed by water flow velocity, event duration and water contamination (Kelman and Spence 2004). Building characteristics with greatest influence on flood damage include building type, building material and whether flood proofing measures are in place (Kreibich *et al.* 2005 and Merz *et al.* 2013).

The most common flood vulnerability model type is a stage-damage curve, which is a damage function relating flood water depth to consequences. Models are commonly categorised according to how they are established (see e.g. Smith 1994); empirical models are derived from consequences data from past events through regression, or comparable methodologies. Conversely, synthetic models are established through “what-if” analyses on hypothetical buildings and expert judgment. Advantages and disadvantages of both approaches are summarised in Merz, Kreibich *et al.* (2010). A further distinction is made between absolute models, where consequences are given in absolute monetary terms (see e.g. Penning-Rowsell *et al.* 2010) and relative models, where consequences are expressed as a fraction of receptor value (see e.g. Scawthorn, Blais *et al.* 2006). While the former is generally easier to estimate, the latter has the advantage of better transferability in time and space (Merz, Kreibich *et al.* 2010). Furthermore, models are available at different levels of spatial aggregation to match the aggregation level of receptors (Messner *et al.* 2007). Macro- and meso-scale models generally specify consequences per areal unit, in some cases with consideration of land cover information (e.g. Thieken *et al.* 2008a). These models do not consider individual receptors, but instead provide aggregated estimates of direct consequences, e.g. for administrative units. Conversely, micro-scale models allow determination of direct consequences to individual receptors, therefore enabling detailed consideration of building characteristics and local flood characteristics.

Empirical damage functions seem to be the most common type of vulnerability model. Their popularity may be explained by the straightforward derivation of the model from consequence data. However, these models are limited in several regards. First, empirical damage functions lack transferability in time and space, see Cammerer *et al.* (2013). Second, despite efforts to improve consequence data quality (Elmer 2012), damage data from past events underlying vulnerability models often lacks detail. Finally, damage data sample size is often too small to reliably infer damage functions for different connotations of each building characteristic and hazard index.

Several of these concerns are addressed by synthetic vulnerability models; they are more flexible and allow for systematic accounting of building characteristics and hazard characteristics. A prominent example is found in the Multi-Coloured Handbook (Penning-Rowsell *et al.* 2010). Based on seminal work by the same authors from the last decades (e.g. Penning-Rowsell and Chatterton 1977), the Multi-Coloured Handbook characterises buildings as collections of components and estimates monetary losses for individual building components in function of water depth and the duration of flood.

The monetary loss to the building is then found by summing up component losses. Although the modelling approach is generally valid, the transferability of the resulting damage functions to other countries is still limited, since consequences are expressed in absolute monetary terms and the modelled building configurations are UK proprietary.

A further dimension of vulnerability modelling approaches is found in process-based vulnerability models, which aim at understanding and reproducing the actual damage processes; see Papathoma-Köhle *et al.* (2010) for an overview, and Mazzorana *et al.* (2014), Kelman (2002), Caraballo-Nadal *et al.* (2006) and De Risi *et al.* (2013), for examples. Process-based models often utilise the fragility concept described in Section 2.4.4. The first step in establishing a process-based vulnerability model is to identify relevant failure mechanisms and damage process; for residential buildings, a list of relevant failure mechanisms is found in Kelman and Spence (2003a). Then, an engineering model and limit state functions are established for each failure mechanism, allowing for calculating the probability of a building being in a certain damage state for a given hazard indices. Process-based fragility models are the most sophisticated vulnerability modelling approaches, as they allow for detailed consideration of building characteristics and can be readily improved when new knowledge on damage processes becomes available. However, the quality of the model hinges on good understanding of damage processes, which is not always available. The need for a better understanding of damage processes is emphasised in Meyer *et al.* (2013).

### 3.4.2 Modelling framework and modelling choices

Numerous comprehensive flood risk assessment models and methodologies are available. Examples of non-commercial models include HAZUS (Scawthorn, Blais *et al.* 2006, Scawthorn, Seligson *et al.* 2006) and the model by Dutta *et al.* (2003). Commercial models include solutions by Risk Management Solutions (RMS 2014) and Air Worldwide (AIR 2014), which are mainly employed for risk assessment in the insurance industry. No comprehensive list of flood risk assessment methodologies can be provided here; instead, an attempt is made to highlight modelling choices required when selecting or developing a flood risk assessment methodology.

A flood risk assessment methodology necessitates a source, pathway, receptor and consequence model. The choice of models is a prerogative of the risk modeller and should be made considering boundary conditions, such as spatial scope, available data, available computational resources and the aim of the risk assessment. A modelling

methodology also must be selected, e.g. Monte Carlo simulation or Bayesian Probabilistic Network, which both allow for consistent consideration and propagation of uncertainty.

In a common modelling approach, flood risk is approximated by simulating a set of flood events with different magnitudes and probability of occurrence. For each event, hazard characteristics are assessed and, from these, consequences are derived, possibly with consideration of uncertainty. The joint consideration of all events in the event set provides an approximation of flood risk. This is a sound approach; however, it often requires a large number of flood event simulations, which can be computationally expensive. The most expensive part of modelling is generally flow routing, and, as a consequence, the choice of flow routing model has implications for the rest of the model, as explained in the following. When a lumped flow routing model is chosen, an individual simulation is not very accurate (large model uncertainty); however, it becomes possible to simulate a large number of flood events. This allows, for instance, explicit consideration of hazard uncertainty and accounting for multiple protection structure breaching scenarios. Conversely, when a distributed flow routing model is chosen, each simulation is more accurate, but the number of events that can be simulated becomes smaller.

Choice of vulnerability and fragility models is driven by the risk assessment goal. When risk assessment is utilised for decision analysis on protection structure optimisation or flood proofing of a building, the vulnerability model or fragility model of the structure must accommodate decision alternatives. Conversely, the accommodation of decision alternatives is not a requirement in risk assessment in an insurance context

Ideally, receptor models, hazard models and consequence models are available on the same spatial scale. When they are not, it becomes necessary to spatially up-scale or down-scale models and/or data, to have matching resolutions of all elements in the risk assessment. Upscaling may involve aggregation of data and models; when input data and models are available at micro-scale, the change of model and data to meso- or macro-scale generally involves only statistical aggregation. For the aggregation to be correct, it is important to consistently consider uncertainty and dependencies from common-cause effects and epistemic uncertainty (Faber *et al.* 2007). Downscaling (or disaggregation) of models and data is inherently more challenging, since it involves moving from less detailed to a more detailed characterisation. It can be modelled based on statistical relationships, or based on underlying processes. Whichever approach is

chosen, it is important to consider and represent the uncertainty introduced in the downscaling process.

### 3.5 *Flood risk management measures*

Based on flood risk assessment, efficiency of different flood risk management measures can be evaluated using principles of risk-based decision analysis introduced in Section 2.5. In the following, an overview of available risk management measures is provided.

Flood risk management measures are generally categorised as structural and non-structural. Most structural measures aim at restraining the flood waters from reaching a receptor, i.e. by keeping the water in the river bed, or by directing it to areas with low receptor value density. Such structures are available with a large range of costs, spatial scopes, protection height and structural resistance. They include different types of dams, dikes and local flood barriers. Another type of structural measures aims at reducing the vulnerability of receptors, e.g. by implementing water-proof materials. Non-structural measures are very diverse and include education and risk awareness raising in the population, implementation of early-warning systems, event management plans and post-event recovery management plans (see Jha *et al.* 2012). An overview of different flood risk management measures is provided in the following.

#### 3.5.1 *Large scale structural measures*

Large structural measures include structures built across a river (i.e. dams and weirs), or along rivers (i.e. dikes and river walls). Structures built across rivers are often built for purposes other than flood risk management, i.e. energy production, to render rivers navigable, control river erosion and sediment transport. Nevertheless, such structures can be beneficial for flood risk management as they allow regulation of river discharge and possibly reduction of peak river discharge during a flood event. In contrast, dikes along rivers are specifically built for flood risk management; their design rationale is linked to the reduction of flood risk. As previously mentioned, dike design was traditionally based on a single design event (e.g. 100 year event), from which the structure should offer protection. Most structures built in the 20<sup>th</sup> century were built according to such criteria. Although the choice of design event may have included consideration of potential consequences, this approach generally did not give detailed consideration to the probability of a protection structure failure and its consequences.

More sophisticated design approaches consider the reliability of a dike structure, considering and possibly optimising, its probability of failure in a reliability-based approach (Voortman and Vrijling 2005). When dike reliability is considered and optimised together with the consequences of a failure, the design approach becomes risk-based (Voortman 2003).

A dike reliability analysis starts with the identification of relevant failure mechanisms; see Morris *et al.* (2009) for an overview. Failure mechanisms of river dikes include overtopping, piping, inner and outer slope failure and erosion; see van Mierlo *et al.* (2007) for further detail and an illustration. According to ASCE/EWRI (2011), surface erosion due to overtopping and internal erosion by piping are the most common failure mechanisms. From each failure mechanism, a breach can initiate and, if the hydraulic load is large, gradually grow. As such, dike failure modelling includes two further steps: probability assessment of a breach initiating and assessment of the breach development. Fragility models for different types of dikes are widely discussed in literature; a state of the art is provided in Allsop *et al.* (2007). Other fragility models are found in Kingston *et al.* (2011), Vorogushyn *et al.* (2009), Buijs *et al.* (2009) and Voortman and Vrijling (2005).

### 3.5.2 *Local scale structural measures*

Local scale structural measures comprise several barriers to prevent or reduce flood hazard in small areas (such as villages or individual receptors), or to temporarily raise the protection height of existing large scale structural measures. A thorough summary of available local flood barriers, including advices for selecting and operating them, is available in Ogunyoye *et al.* (2011). Local flood protection measures include temporary and demountable barriers, with a sand bag wall as the most prominent example of a local flood barrier (Reeve *et al.* 2003). Temporary barriers include a range of solutions: air filled and water filled tubes, filled containers, freestanding barriers and frame barriers. Demountable barriers differ from temporary barriers in that they are partially preinstalled, with an anchoring system or foundations required to deploy the barrier.

Local flood barriers differ greatly in protection height, which can range from few dozen centimetres to several meters and structural resistance (Bramley and Bowker 2002). Each barrier type is subject to different failure modes, which may include overtopping, breaching, collapsing, excessive seepage, puncturing and sliding. Vulnerability models for local flood barriers are not widely available in literature.

However, commercial barrier vendors should generally have analytical vulnerability models and/or empirical results of structural tests available.

Whereas most flood risk management measures can be designed and selected with only risk reduction in mind, the selection of local flood barrier requires consideration of the flood lead time, i.e. the time available between flood warning and the actual flood. If the lead time is very short, a pre-installed barrier must be selected.

### 3.5.3 *Micro scale structural measures*

Micro scale structural measures are directly applied to buildings and are often called flood proofing measures. While local protection structures aim at keeping the flood water away from buildings, flood proofing measures aim at reducing consequences when flood water reaches the building, by hindering water infiltration into the building or by reducing susceptibility of building components to water contact.

An overview of flood proofing measures for residential building is given in Elliott and Leggett (2002), with several flood proofing measures often combined in one flood proofing strategy. Typical flood proofing strategies are (Zevenbergen *et al.* 2007):

- **Dry-proofing** – A dry-proofing strategy aims at hindering the water from entering the building by closing all possible water infiltration routes (e.g. air bricks, ventilation shafts, door cracks, windows cracks and sewer system) up to ca. 1m over terrain. Dry-proofing above this threshold is not recommended as it may favour hydrostatic pressure on the building envelope, which can, in turn, compromise the structural integrity of the building. Dry-proofing can be achieved with both temporary and permanent measures (Joseph 2014).
- **Wet-proofing** – Wet-proofing allows for flood water entering the building; however, it aims at reducing the susceptibility of building components and contents to water contact. This includes employing water resistant materials and placing susceptible installations and contents in elevated positions.
- **Avoidance** – The avoidance strategy entails constructing buildings so that water can hardly reach them, i.e. on top of stilts, columns or walls.

Practical instructions on selection and implementation of flood proofing measures are found in Bramley and Bowker (2002), Bowker (2002) and Wingfield *et al.* (2005).

Effectiveness of each strategy depends on local hazard characteristics, i.e. on the frequency and intensity of hazard actions impacting a receptor.

Benefits of flood proofing are conceptually described in Naumann *et al.* (2011) which provides vulnerability curves for buildings with and without flood proofing measures. In Thielen *et al.* (2005) and Kreibich *et al.* (2005) the benefits of flood

proofing are illustrated by comparing damage data from flood proofed buildings and from buildings without flood proofing. Bubeck *et al.* (2012) compares damages from the 1993 and 1995 flooding of the Meus to identify the effectiveness of flood proofing measures applied between the two events.

Cost-benefit analyses are found in several publications, e.g. Joseph (2014), Zevenbergen *et al.* (2007), Gersonius *et al.* (2008) and Kreibich *et al.* (2011), each taking a different approach to the estimation of costs and benefits. For instance, Kreibich *et al.* (2011) estimates the benefits from damage data and costs from actual expenses that might incur when implementing a flood proofing measure. Joseph (2014) estimates both costs and benefits (including reduction of intangible consequences) through a survey. Zevenbergen *et al.* (2007) and Gersonius *et al.* (2008) estimate benefits through an analytical calculation of vulnerability curve (albeit without providing modelling details) and costs seem to be postulated.

Several sources, e.g. Kelman (2007) and Joseph (2014), underline the importance of considering intangible damages in any cost-benefit analyses for flood proofing of buildings.

#### 3.5.4 *Non-structural measures*

Non-structural measures help manage consequences when a flood event occurs and are often employed to manage residual risk of structural measures.

Non-structural measures are categorised by Jha *et al.* (2012) as increased preparedness, flood avoidance, emergency planning and management, recovery speed-up and using recovery to increase resilience. Several measures are listed and detailed in the following.

Increased preparedness includes measures such as risk awareness (education about the possibility of floods occurring), health awareness (education about the health risk of floods, e.g. drinking water contamination, faecal contamination of flood water or decaying human/animal remains). These measures help population and authorities to react appropriately in the event of a flood.

A typical flood avoidance measure is spatial planning and zoning, which allows for limiting urban development in flood zones (see e.g. Poussin *et al.* 2012).

Flood insurance allows the transferral of financial flood risk from a property or business owner to an insurance company (Swiss Re 2007). Insurance is increasingly recognised as beneficial for risk management, as the risk assessment of the insurance

company may raise risk awareness, thus encouraging the implementation of risk reduction measures (Jha *et al.* 2012, Poussin *et al.* 2013). According to Thieken *et al.* (2007) raised risk awareness has a similar positive impact on the implementation of flood proofing measures.

Emergency planning and management aims at facilitating an optimal emergency response to minimise flooding consequences and allow for a prompt post-event recovery. As part of an emergency plan, an early warning system is generally put in place to give advance notice of an imminent flood, allowing emergency plans to be put into action and vulnerable members of society to be evacuated. Emergency planning may include the organisation of first aid and rescue activities, continuity planning of government activity and critical infrastructure (Jha *et al.* 2012).

Lastly, non-structural risk management measure may include plans for recovery and reconstruction of areas affected by floods. Here, the focus should be on a speedy recovery of infrastructure functionality, as well as state of the art reconstruction of destroyed and damaged structures. This includes integrating lessons learned from the flood event, but also incorporating technological advances in building design, energy efficiency, transportation infrastructure, telecommunication infrastructure and urban planning. Through these steps, the negative consequences of the flood event can be partially offset by gains in efficiency and life quality that come from improved buildings and infrastructure.

## **4 Hierarchical flood protection system**

A definition and characterisation of hierarchical flood protection system is provided and a modelling approach is proposed both for flood risk analysis in the presence of a hierarchical flood protection system and to identify its optimal configuration. The chapter concludes with possible challenges faced when modelling a hierarchical flood protection system.

### 4.1 Introduction

Structural flood risk management measures, here referred to as flood protection structures, reduce flood risk by restraining flood water from reaching receptors. As described in Section 3.5, flood protection structures are available for a range of spatial scales; examples are dams, dikes and local flood barriers. Moreover, if a dry-proofing strategy is applied to a building, its building envelope is considered a flood protection structure for the building interior.

Flood protection structures reduce risk; however, since they can fail, they always entail residual risk. This observation directly indicates the value of having several hierarchically integrated flood protection structures on the pathway between source and receptor, where each additional hierarchy level allows for a further reduction of flood risk. When two or more flood protection structures are sequentially integrated on different spatial scales, the flood protection system becomes hierarchical. Figure 4.1 schematically illustrates a hierarchical flood protection system with four hierarchy levels.

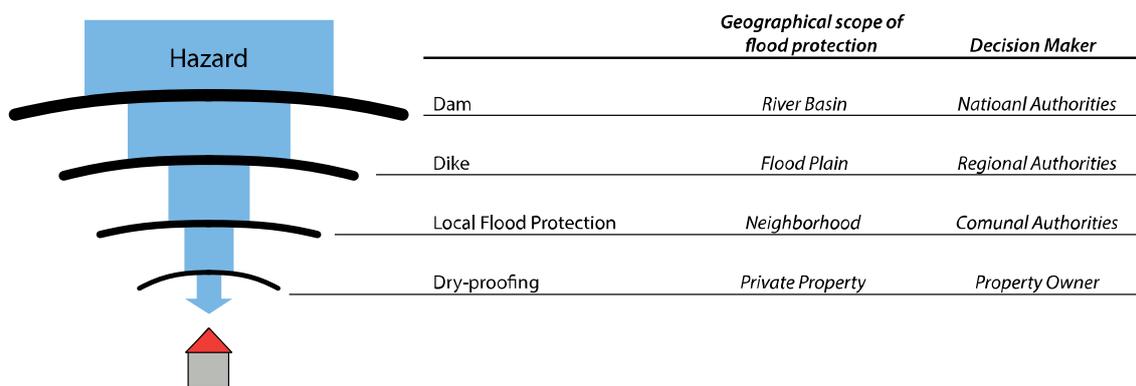


Figure 4.1 - Schematic illustration of a hierarchical flood protection system with indication of geographical scope and decision maker for each structure

Figure 4.1 illustrates a hierarchical flood protection system for a single receptor; however, large protection structures generally offer protection to more than one building. This is illustrated in Figure 4.2, where a hierarchical flood protection system with several receptors is schematically illustrated. Whereas on the top hierarchy level there is generally only one protection structure, at lower hierarchical levels more protection structures can be present. Also, as illustrated in the figure, some receptors may be protected by more protection structures than others.

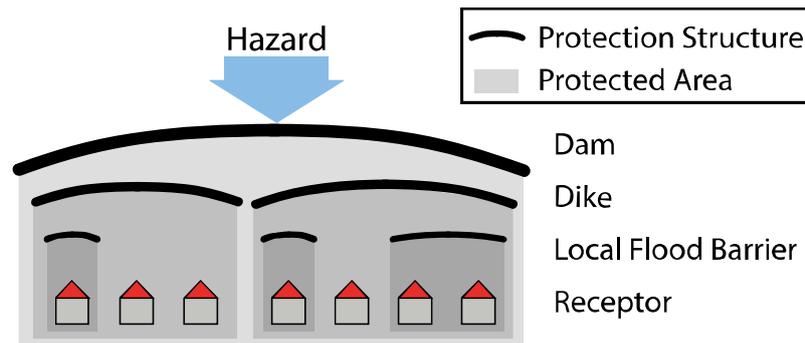


Figure 4.2 - Hierarchical flood protection system illustrated for several receptors

When optimised, hierarchical flood protection systems generally allow for a larger expected utility than single-structure protection systems. The reason for a hierarchical flood protection system to be associated with larger expected utility are mathematically trivial, with each additional hierarchy level considered, the decision maker has a larger set of decision alternatives to choose from, which implies that the expected utility of the optimal decision alternative is non-decreasing with each additional hierarchy level.

Literature recognises the value of managing flood risk by integrating a number of different structural and non-structural flood risk management measures (e.g. Hall, Dawson *et al.* 2003; Dawson *et al.* 2004; Jha *et al.* 2012). The similar concept of multiple layer of defence and multi-line safety is found in Tsimopoulou *et al.* (2012), Tsimopoulou *et al.* (2013), Lopez (2006), Jongejan *et al.* (2012). An exception is found in Nehlsen *et al.* (2007), where a system of cascading (hierarchically integrated) dikes is described. The

In the following, a characterisation of hierarchical flood protection system is provided before their modelling is examined.

#### **4.2 Characteristics of a hierarchical flood protection system**

Several characteristics of hierarchical flood protection systems are introduced; notably, advantages of hierarchical flood protection systems compared to single-structure protection systems are synthesised as: improved risk reduction, tailoring, robustness and flexibility. Furthermore, several cautionary observations are made, to highlight challenges potentially entailed in a hierarchical flood protection system.

4.2.1 Risk reduction

In a hierarchical flood protection system, individual flood protection structures jointly reduce risk; however, the contribution to risk reduction differs by structure type. Large protection structures, such as dams and dikes effectively reduce risk from both small and large flood events. Smaller protection structures, such as local flood barriers and flood proofing measures, effectively reduce risk from small flood events; however, they are generally ineffective against large flood events. The hierarchical integration of several protection structures offers protection against small, as well as large flood events; for the latter, it reduces residual risk. Figure 4.3 schematically exemplifies these observations by illustrating the risk reduction for different protection systems and flood events of different magnitude<sup>8</sup>.

These observations illustrate that the aim of hierarchical flood protection systems is not redundancy in the sense of having two or more structures with the same role (quite obviously, dike and local flood barriers cannot provide the same protection).

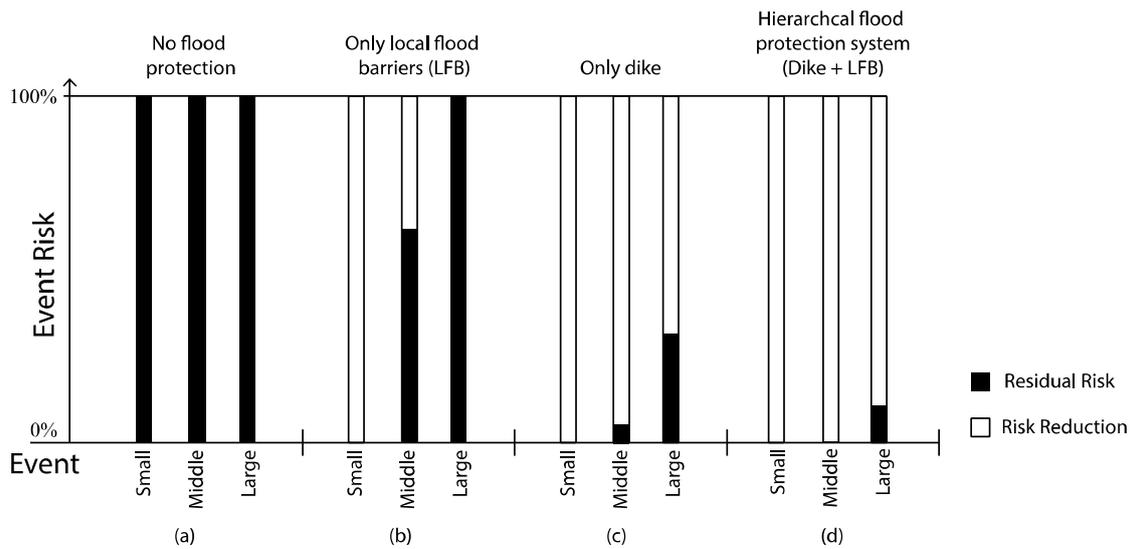


Figure 4.3 - Schematic illustration of the risk reduction achieved by local flood barriers (b), a dike (c), and a hierarchical flood protection system (d). Each bar corresponds to one flood event. For each flood protection system risk reduction and residual risk are illustrated for three hypothetical events (small event, middle event and large event)

<sup>8</sup> Note that Figure 4.3 is a schematic illustration of how risk is reduced for different protection systems; it is, without doubt, possible to design a dike which achieves a similar risk reduction as the hierarchical flood protection system, however, in all likelihood, at a higher cost.

4.2.2 Tailoring

Hierarchical flood protection systems allow for tailoring of flood protection according to risk reduction requirements dictated by hazard characteristics and receptor characteristics. By hierarchically integrating different protection structures, individual protection structures may be designed to effectively complement each other. For instance, the extensive protection of a dike can be complemented with focused protection of local flood barriers where the receptor value density is high.

For illustration, a flood plain with one village and extensive agricultural land is considered in Figure 4.4. In such a system, the higher receptor value density in the village would generally command a higher hazard reduction than agricultural land. When a single protection structure is responsible for flood risk reduction, it has to be built to meet the target risk reduction in the village (Figure 4.4, top). The surrounding agricultural land will profit from the same hazard reduction, and while this is welcome, it is probably not economically justifiable. Conversely, in a hierarchical flood protection system (Figure 4.4, bottom) the target hazard reduction for the village is jointly assured by a dike and local flood barriers. The dike can be built to a lower protection height and hazard reduction for agricultural land is smaller as a result, which is reasonable from an economic standpoint. In summary, a hierarchical flood protection system protects areas with a high receptor value density, without overprotecting areas with low receptor value density.

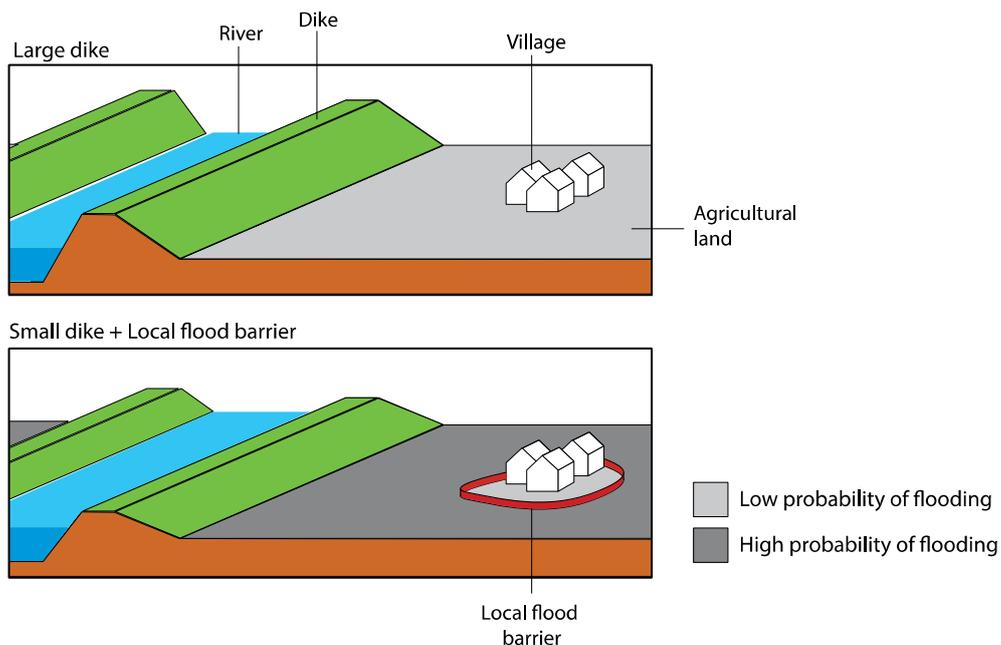


Figure 4.4 - Comparison between two flood protection systems. On top, receptors and the whole flood plain are protected by a large dike; at the bottom, a smaller dike protects the flood plain; areas with high asset value concentration are additionally protected by a local flood barrier

### *4.2.3 Robustness*

A further advantage of hierarchical flood protection system is an increase in protection system robustness. A system is considered robust when the indirect consequences of its failure are commensurate to the direct consequences of its failure (see JCSS 2008). Failure in a flood protection system occurs when a protection structure is either breached or overtopped. When this happens in a single-structure protection system, receptors are directly exposed to flood water without any further protection. Conversely, in the presence of a hierarchical flood protection system, the failure of one structure is less consequential, since additional protection structures may still be in place. Therefore, a hierarchical flood protection system is generally expected to be more robust than a single-structure protection system.

### *4.2.4 Flexibility and capacity for future changes in flood risk*

As described in previous chapters, future flood risk is subject to large uncertainty, e.g. due to climate change, which may cause future flood hazard increase or decrease. This uncertainty puts decision makers in a difficult position when planning large flood protection structures, which can take up to a decade for planning and construction, and have an expected life time of up to a century. Given the substantial uncertainty, protection height of a protection structure can easily be over- or under-designed in regard to future flood risk. Both cases amount to a waste of societal resources, and should be avoided. Furthermore, if decision makers realise that a dike is under-designed, time required to increase protection height of a dike is generally long, leaving receptors with insufficient protection for a significant amount of time.

This challenge might be alleviated through consideration of hierarchical flood protection systems, as their adaption to changing flood risk is easier; e.g. it is possible to improve flood protection by adding an additional hierarchy level or upgrading protection structures on a subordinate hierarchy level. Furthermore, the decision maker can consider the flexibility of hierarchical flood protection systems in the planning phase of large flood protection structures. That is, faced with large uncertainty in regard to future risk, decision makers may plan the flood protection system already considering future “risk reduction capacity”, i.e. potential future second or third hierarchy levels, which could be implemented in case future flood risk requires it.

#### 4.2.5 *Optimisation of hierarchical flood protection system*

Three main benefits of hierarchical flood protection system have been identified in optimised risk reduction through tailoring, increased robustness of the flood protection system, and increased flexibility for future flood risk reduction.

When optimizing a flood protection system with a formal decision analysis, the decision maker may consider all the above benefits in a multi-objective utility function. However, he may also plan a hierarchical flood protection system with focus on one benefit, as it is generally not possible to optimise all benefits simultaneously. Regardless of preferences and aims of the decision maker, a flood risk assessment should always be the basis for decision analysis.

#### 4.2.6 *Cautionary notes*

Flood protection through a hierarchical flood protection system seems to be beneficial; however, particular attention is required in its planning and implementation, especially when it is optimised for risk reduction. Compared to a single-structure flood protection system, such a system may deliberately accept a higher probability of flooding for certain areas, typically areas with low receptor value density. This may be economically sensible, since the costs for flood risk reduction cannot be justified. However, it becomes of paramount importance to accurately model all aspects of flood risk. In particular, a deliberate increase in probability of flooding (relative to a single-structure protection system) can be justified only if all types of consequences (cf. Figure 2.3) are considered.

A second aspect requiring careful attention in a hierarchical flood protection system is the execution of flood risk management. Flood protection structures at different scales are, in all likelihood, managed by different authorities. The good functioning of the protection system hinges on correct execution of pre-event measures and event management on each hierarchy level.

In summary, hierarchical flood protection system offer benefits compared to single-structure protection system, but at the same time they require flawless planning and execution.

### 4.3 Flood risk assessment

In this section a mathematical formalisation of hierarchical flood protection system is proposed along with a methodology for flood risk analysis in the presence of a hierarchical flood protection system.

#### 4.3.1 Hazard assessment in a hierarchical flood protection system

When receptors are protected by one or more flood protection structures, flooding at the location of a receptor only occurs when all flood protection structures protecting the receptor are overtopped and/or breached. Hazard impacting a receptor is not only a function of the flood event itself, but also of the state of each protection structure protecting the receptor. Therefore, when modelling hazard impacting a receptor, hazard models and fragility models of protection structures need to be jointly considered (as illustrated in Figure 4.5). The fragility assessment of flood protection structures can be considered integral part of hazard assessment.

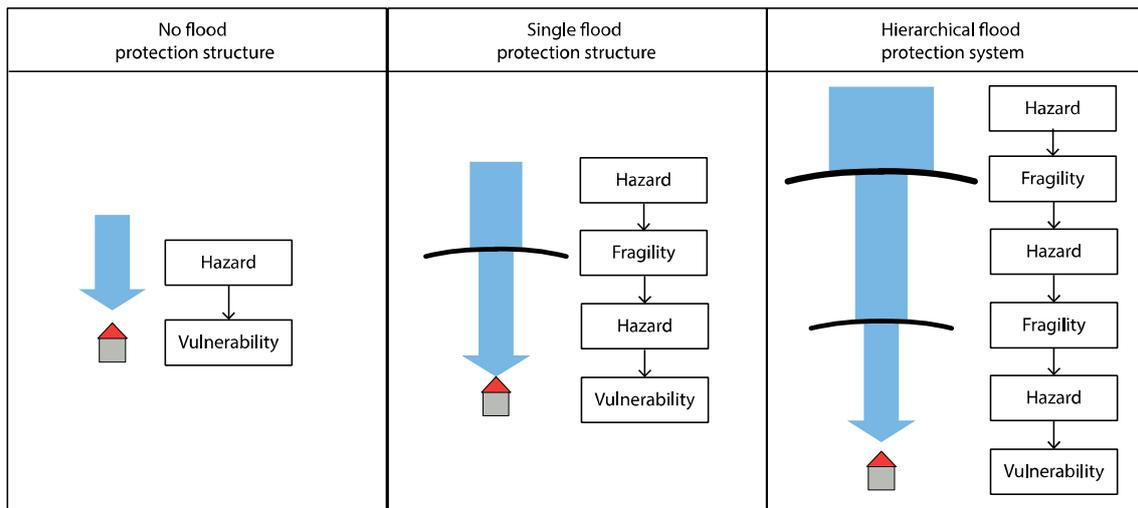


Figure 4.5 - The hazard impacting a receptor is calculated by alternatively considering hazard and fragility models

#### 4.3.2 Methodology for flood risk assessment

A hierarchical flood protection system is an integration of two or more flood protection structures on different spatial scales, which jointly contribute to flood risk reduction. Figure 4.6 schematically illustrates a generic hierarchical flood protection system. The hierarchy levels are numbered from  $i=1$  for the most upstream level (closest to flood source), to  $i=N$  for the most downstream level. Each hierarchy level may entail several protection structures (cf. Figure 4.2).

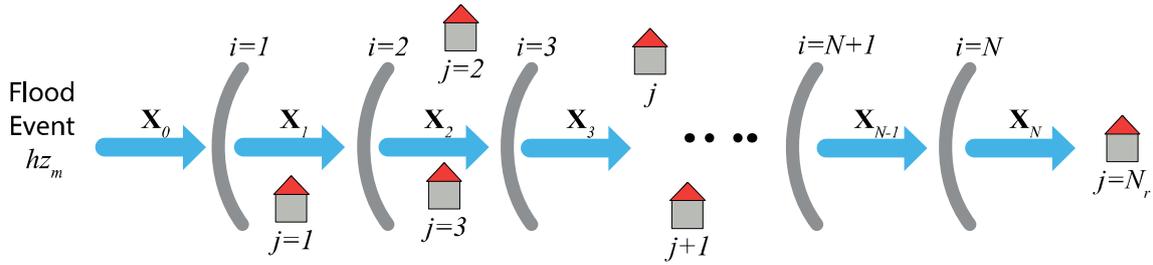


Figure 4.6 - Hierarchical flood protection system: nomenclature used for the formalisation

Flood events are random phenomena here approximated with a finite number of sample events. The sample space of flood events affecting a study area<sup>9</sup> is defined as  $\Omega_{HZ} = \{hz_m, m = 1, 2, \dots, M\}$ , where  $hz_m$  is a sample hazard event and  $M$  is the number of samples in  $\Omega_{HZ}$ . Hazard event  $hz_m$  is characterised by parameter  $\varphi_{HZ}$ ;  $\varphi_{HZ}$  is generally a physical parameter describing flood event characteristics at the hazard source, e.g. river discharge. In principle,  $\varphi_{HZ}$  may also be a vector of several parameters. The occurrence probability of  $hz_m$  is  $p_{hz_m}$ . As always, the occurrence probability of a hazard event is related to a reference time period, generally 1 year in natural hazard risk assessments.

Hazard events  $hz_m \in \Omega_{HZ}$  may be utilised as an input for hazard assessment, by which the probabilistic characterisations of hazard indices throughout the studied system is assessed. To allow for a general representation of hazard indices, the study area is divided into different sections, each delimited by a hierarchy level. Hazard indices upstream of the first protection structure ( $i=1$ ) are modelled by a random vector  $\mathbf{X}_0$ . Hazard between the  $i$ th and  $(i+1)$ th hierarchy levels is modelled by random vector  $\mathbf{X}_i, i=1, \dots, N$ . Random vector  $\mathbf{X}_i, i=0, \dots, N$  may include the characterisation of development in time and space of water depth, water flow velocity and other hazard indices between the  $i$ th and  $(i+1)$ th hierarchy levels. Parameters necessary to model  $\mathbf{X}_i$  are summarised in the parameter vector  $\varphi_{\mathbf{X}_i}$ , and may include terrain geometry, river bathymetry and surface friction.

Random state variable  $S_i, i=1, 2, \dots, N$  characterises the state of all protection structures at the  $i$ th hierarchy level. Variable  $S_i$  includes information on whether protection structures are structurally intact or breached, and in the latter case, the breaching location(s). Relevant characteristics of all flood protection structures

<sup>9</sup> Study area is understood as the geographical region for which the flood risk assessment is carried out, e.g. a village, commune, flood plain or river basin.

(geometry, resistance and location) at the  $i$ th hierarchy level are characterised by parameter vector  $\boldsymbol{\varphi}_{S_i}$ .

Receptors in the study area are indexed by  $j=1,\dots,N_r$ , where  $N_r$  is the total number of receptors. Direct consequences to receptor  $j$  are modelled by random variables  $L_j, j=1,\dots,N_r$ ; parameters necessary to model  $L_j$  are summarised by parameter vector  $\boldsymbol{\varphi}_{L_j}$  (receptor value, location and vulnerability parameters). Receptors are considered to be randomly distributed on the study area. As such, different receptors may be protected by a different number of protection structures as illustrated in Figure 4.2. For receptor  $j$ , located between the  $i$ th and  $(i+1)$ th hierarchy levels, the relevant hazard indices are given in  $\mathbf{X}_i$ . To unequivocally identify the hazard impacting receptor  $j$ , it is identified with  $\tilde{\mathbf{X}}_j$  which is equal to  $\mathbf{X}_i$ .

Note that parameters in  $\boldsymbol{\varphi}_{HZ}, \boldsymbol{\varphi}_{S_i}, \boldsymbol{\varphi}_{X_i}, \boldsymbol{\varphi}_{L_j}$  may also be considered as random variables, e.g. to consider statistical uncertainty: Then, a formulation of the respective probability density functions becomes necessary to assess flood risk.

Flood risk assessment aims at determining flood risk for all receptors  $j=1,\dots,N_r$  in the study area. In a general approach, this requires formulation of the joint probability density function of all introduced variables. A direct formulation of the joint probability density function with a large number of variables and dependencies is cumbersome and impractical. Therefore, assumptions are made allowing for a formulation of the joint probability density function as the product of conditional probability density functions. In particular the following assumptions are made:

- Hazard variables  $\mathbf{X}_0$  are only conditional on hazard event  $hz_m$ .
- Hazard variables  $\mathbf{X}_i$  at the  $i$ th hierarchy level are only conditional on hazard variables  $\mathbf{X}_{i-1}$  and state variable  $S_{i-1}$  at the forgoing hierarchy level  $i-1$ .
- State variable  $S_i$  at the  $i$ th hierarchy level is only conditional on hazard variables  $\mathbf{X}_{i-1}$  at the forgoing hierarchy level  $i-1$ .
- Direct consequence  $L_j$  is only conditional on hazard variables  $\tilde{\mathbf{X}}_j$ .

From these assumptions the following formulations follow. Hazard  $\mathbf{X}_0$  is modelled conditionally on hazard event  $hz_m$  and has conditional probability density function  $f_{\mathbf{X}_0|hz_m}(\mathbf{x}_0 | hz_m; \boldsymbol{\varphi}_{\mathbf{X}_0})$ . The state  $S_i$  of protection structures on the  $i$ th hierarchy level is modelled conditionally on  $\mathbf{X}_{i-1}$ , and has conditional probability density function  $f_{S_i|\mathbf{X}_{i-1}}(s_i | \mathbf{x}_{i-1}; \boldsymbol{\varphi}_{S_i})$ . Hazard vector  $\mathbf{X}_i$  is modelled conditionally on  $\mathbf{X}_{i-1}$  and  $S_i$ , and has conditional probability density function  $f_{\mathbf{X}_i|\mathbf{X}_{i-1}, S_i}(\mathbf{x}_i | \mathbf{x}_{i-1}, s_i; \boldsymbol{\varphi}_{\mathbf{X}_i})$ . Lastly, direct

consequences  $L_j$  to receptor  $j$  are modelled conditionally on hazard variables  $\tilde{\mathbf{X}}_j$ , and have conditional probability density function  $f_{L_j|\tilde{\mathbf{X}}_j}(l_j|\tilde{\mathbf{x}}_j;\Phi_{L_j})$ .

Conditional probability density functions  $f_{\mathbf{x}_0|hz_m}(\mathbf{x}_0|hz_m;\Phi_{\mathbf{x}_0})$  and  $f_{\mathbf{x}_i|\mathbf{x}_{i-1},S_i}(\mathbf{x}_i|\mathbf{x}_{i-1},S_i;\Phi_{\mathbf{x}_i})$  are typically determined with a flow routing model;  $f_{S_i|\mathbf{x}_{i-1}}(S_i|\mathbf{x}_{i-1};\Phi_{S_i})$  with protection structure fragility models and  $f_{L_j|\tilde{\mathbf{X}}_j}(l_j|\tilde{\mathbf{x}}_j;\Phi_{L_j})$  with receptor vulnerability models.

The joint probability density function may be obtained by multiplying the conditional probability density function for all introduced variables; its formulation is however not necessary for this application. Instead the conditional probability density functions are successively combined and variables which are not explicitly necessary for the risk calculation are successively marginalized.

First, the joint probabilistic characterisation of  $\mathbf{X}_0, \mathbf{X}_1, \dots, \mathbf{X}_N$  given hazard event  $hz_m$  is obtained through the chain rule and marginalisation of the state variables  $S_i, i = 1, \dots, N$ <sup>10</sup>:

$$f_{\mathbf{x}_0, \mathbf{x}_1, \dots, \mathbf{x}_N|hz_m}(\mathbf{x}_0, \mathbf{x}_1, \dots, \mathbf{x}_N|hz_m) = \int \dots \int_{D_{S_i}, i=1, \dots, N} f_{\mathbf{x}_0|hz_m} \prod_{i=1}^N (f_{\mathbf{x}_i|\mathbf{x}_{i-1}, S_i} f_{S_i|\mathbf{x}_{i-1}} ds_i). \quad (4.1)$$

where  $D_{S_i}$  is the domain of  $S_i$ .

Based on the joint probability density function of all hazard variables in Equation (4.1), in the next modelling step direct consequences to receptors are characterised, first for an individual receptor  $j$ , thereafter jointly for all receptors  $j = 1, \dots, N_b$ . The probability density function of direct consequences  $L_j$  given flood event  $hz_m$  is modelled as:

$$f_{L_j|hz_m}(l_j|hz_m) = \int_{D_{\tilde{\mathbf{x}}_j}} f_{L_j|\tilde{\mathbf{X}}_j} f_{\tilde{\mathbf{x}}_j|hz_m} d\tilde{\mathbf{x}}_j, \quad (4.2)$$

where  $D_{\tilde{\mathbf{x}}_j}$  is the domain of  $\tilde{\mathbf{X}}_j$ ,  $f_{\tilde{\mathbf{x}}_j|hz_m}(\tilde{\mathbf{x}}_j|hz_m)$  is obtained from Equation (4.1) through a variable substitution  $\mathbf{X}_i = \tilde{\mathbf{X}}_j$  and marginalisation of all other hazard variables. The joint probability density function of  $L_j, j = 1, \dots, N_r$  given flood event

<sup>10</sup> Note that, to allow a compact notation in Equation (4.1) and following, the round brackets of probability density functions are omitted on the right side of equations.

$hz_m$  is modelled as:

$$f_{L_1, L_2, \dots, L_j | hz_m}(l_1, l_2, \dots, l_j | hz_m) = \int \dots \int_{\mathbf{x}_i, i=0, \dots, N} f_{\mathbf{x}_0, \mathbf{x}_1, \dots, \mathbf{x}_N | hz_m} \prod_{j=1}^{N_r} (f_{L_j | \tilde{\mathbf{x}}_j}) d\mathbf{x}_i \quad (4.3)$$

Direct consequences  $L_{tot}$  to all receptors in the study area are calculated as:

$$L_{tot} = \sum_{j=1}^{N_r} L_j, \quad (4.4)$$

While no general closed form of  $f_{L_{tot} | hz_m}(l_{tot} | hz_m)$  is available, it is fully defined by Equation (4.3) and Equation (4.4). The expected value of  $L_j$  given flood event  $hz_m$  is:

$$E[L_j | hz_m] = \int_{D_{L_j}} l_j f_{L_j | hz_m}(l_j | hz_m) dl_j. \quad (4.5)$$

where  $D_{L_j}$  is the domain of  $L_j$ . The expected value of  $L_{tot}$  given flood event  $hz_m$  is:

$$E[L_{tot} | hz_m] = \sum_{j=1}^{N_r} E[L_j | hz_m]. \quad (4.6)$$

where  $D_{L_{tot}}$  is the domain of  $L_{tot}$ .

In the following, risk  $r_j$  for individual receptors is first characterised, followed by a characterisation of total risk  $r_{tot}$  for all receptors in the study area. In accordance with Equation (2.2), the flood risk for receptor  $j$  is:

$$r_j = \sum_{hz_m \in \Omega_{HZ}} p_{hz_m} E[L_j | hz_m], \quad (4.7)$$

and the total risk for the receptors  $j = 1, \dots, N_r$  is:

$$r_{tot} = \sum_{r_j}^{N_r} r_j. \quad (4.8)$$

This concludes the description of the flood risk assessment methodology in the presence of hierarchical flood protection systems. Several comments on the methodology follow. While the integrals in Equation (4.1) and Equation (4.3) seem prohibitive, in practice, the number of hierarchy levels is often limited to  $N = 4$ .

Furthermore, hazard and vulnerability model are often formulated in a deterministic manner, which significantly simplifies modelling.

The conditional independence assumptions, on which the methodology hinges, seem reasonable; nonetheless, in some instances, they may not hold; e.g. when epistemic uncertainty is explicitly considered. In that case, the presented methodology can be adapted by making all considered probability density functions conditional on epistemic uncertainty variable  $\Xi$ . When its probability density function  $f_{\Xi}(\xi)$  is known,  $\Xi$  can be integrated out in the end result, see Faber *et al.* (2007). Furthermore, in some instances, the conditional independence assumption might not hold for a particular hierarchy level, since hydrostatic and hydrodynamic pressure downstream of the structure may affect its state  $S_i$  (Kelman 2002), or since the hazard upstream of a structure may be reduced when the structure is breached (see e.g. Apel *et al.* 2009). In these cases, on the hierarchical levels in questions the joint probability density function of all dependent variables has to be specified, whereas on all other hierarchy levels the methodology can be maintained as presented.

Implementation challenges related to the flood risk assessment methodology presented are described in Section 4.5.

#### **4.4 Decision analysis for hierarchical flood protection systems**

Based on flood risk assessment, a hierarchical flood protection system can be optimised in accordance with the principles of engineering decision analysis outlined in Section 2.5. Before the formal decision analysis is described, the decision context of a hierarchical flood protection system is briefly discussed.

##### *4.4.1 Decision governance*

When considering flood risk management measures on a range of spatial scales, it is important to recognise that a hierarchical flood protection system not only involves a hierarchy of protection structures, but also a hierarchy of decision makers, all of whom should act in a coordinated manner towards the same goal of managing flood risk in optimal manner. In the words of Dawson *et al.* (2004): "*...decisions take place at a range of levels from national policy strategic planning decisions ..., to local construction, operation and maintenance decisions.*"

While optimal flood risk management should be in the best interest of all involved decision makers, stakeholders might disagree on cost allocation and have different preferences for, or against, development of a particular hierarchy level. For instance, a national government should be interested in optimally protecting its population and assets, but a local commune might have additional preferences, e.g. it might prefer the national government to carry costs for risk reduction measures and, for aesthetic reasons, it might prefer protection structures to be located far away and out of sight. In this thesis, the optimal decision alternative is identified through a normative engineering decision analysis. Utility, however it is defined, is optimised for the whole system, and does not consider particular preferences of individual stakeholders at a subordinate level. Once the optimal decision alternative is identified, it is assumed that all stakeholders enact it, independently of their individual preferences.

#### 4.4.2 Decision alternatives

On each hierarchy level, a number of decision alternatives are available to the decision maker. Decision alternatives available at the  $i$ th hierarchical level,  $i=1,2,\dots,N$ , are defined in set  $A_i = \{a_i^{(0)}, a_i^{(1)}, \dots, a_i^{(K_i)}\}$ , where  $a_i^{(0)}$  identifies the “no action” alternative, i.e. the status quo, and  $K_i + 1$  is the number of decision alternatives considered at the  $i$ th hierarchy level. Decision alternatives for an individual hierarchy level may differ in several regards, i.e. in number and type of flood protection structures, as well as their protection height and structural resistance.

A decision alternative for the whole protection system,  $a_s^{(.)}$ , combines a decision alternative for each hierarchy level  $i=1,2,\dots,N$  and is represented by  $a_s^{(k_1, k_2, \dots, k_N)} = (a_1^{(k_1)}, a_2^{(k_2)}, \dots, a_N^{(k_N)})$ . The set of different decision alternatives for the whole hierarchical flood protection system is:

$$A_s = \{a_s^{(k_1, k_2, \dots, k_N)}, k_i = 1, 2, \dots, K_i, i = 1, 2, \dots, N\}. \quad (4.9)$$

#### 4.4.3 Utility function

To evaluate and compare decision alternatives a utility function must be specified in accordance with principles of risk-based decision analysis outlined in Section 2.5.1. In the context of natural hazard risk management, the utility function should consider all costs and benefit entailed by the prospective risk management measure for the

considered system over the considered time span  $T$ . The benefit of a flood risk management measure is a reduction of flood risk  $r_{tot}$ . The costs associated with a decision alternative  $a^{(k)}$  are modelled by  $c(t, a^{(k)})$ , which captures costs for construction, inspection, maintenance, operation and decommissioning in function of time  $t$ . The expected utility  $E[U(a^{(k)})]$  of decision alternative  $a^{(k)}$ <sup>11</sup> is expressed as:

$$E[U(a^{(k)})] = \sum_{t=0}^T \frac{1}{(1+\rho)^t} \left[ (r_{tot}(a^{(0)}) - r_{tot}(a^{(k)})) - c(t, a^{(k)}) \right], \quad (4.10)$$

where  $U(a^{(k)})$  is the utility of decision alternative  $a^{(k)}$ ,  $a^{(0)}$  is the current state of the protection system and  $\rho$  is the discounting rate. Note that an assumption underlying Equation (4.10) is stationarity of risk  $r_{tot}(a^{(k)})$ ,  $k = 0, 1, \dots, K$  for  $0 \leq t \leq T$ . Furthermore, note that expected utility is calculated relative to risk  $r(a^{(0)})$  entailed in the current protection system. Whereas this is not necessary to rank decision alternatives, it allows for easily identifying decision alternatives with lower expected utility compared to the current protection system, as they would be assigned negative expected utility.

#### 4.4.4 Decision optimisation

All hierarchy levels of a hierarchical flood protection system are jointly optimised to identify the optimal configuration of the hierarchical flood protection system.

Considering a hierarchical flood protection system with  $N$  hierarchy levels, the set of different decision alternatives for the whole protection system is  $A_s = \{a_s^{(k_1, k_2, \dots, k_N)}, k_i = 1, 2, \dots, K_i, i = 1, 2, \dots, N\}$ . The optimal decision alternative  $a_s^* \in A_s$  maximises expected utility in Equation (4.10) and is identified as:

$$E[U(a_s^*)] = \max_{A_s} E[U(a_s^{(k_1, k_2, \dots, k_N)})]. \quad (4.11)$$

where  $E[U(a_s^{(k_1, k_2, \dots, k_N)})]$  is calculated according to Equation (4.10).

<sup>11</sup> Depending on the decision problem,  $a^{(k)}$  can either represent a decision alternative for an individual hierarchy level  $a_i^{(k_i)}$  or a decision alternative for the whole system  $a_s^{(k_1, k_2, \dots, k_N)}$ .

#### 4.5 Challenges in flood risk assessment in the presence of a hierarchical flood protection system

Several challenges, while inherent to flood risk assessment in general, are particularly relevant when considering hierarchical flood protection systems.

A first challenge pertains to consistent modelling of risk at different spatial scales. Considering and optimizing flood protection structures on a range of spatial scales may require flood risk assessment to be carried out at different spatial scales. Here, methodologies for upscaling and downscaling of models and/or data, with consistent consideration of uncertainties and dependencies are necessary.

A second challenge is faced in the computation of the hazard constrained to the area between two hierarchy levels, which is necessary to determine conditional probability density functions  $f_{\mathbf{x}_i|\mathbf{x}_{i-1},S_i}(\mathbf{x}_i|\mathbf{x}_{i-1},S_i;\boldsymbol{\Phi}_{\mathbf{x}_i}), i=1,\dots,N$ . A lumped flow routing model seems well indicated for this modelling step, as it allows subdividing the river catchment into compartments and explicitly model functional relationships between hazard indices in neighbouring compartments. However, as previously mentioned, lumped flow routing models are generally not very accurate. Conversely, the modelling of  $f_{\mathbf{x}_i|\mathbf{x}_{i-1},S_i}(\mathbf{x}_i|\mathbf{x}_{i-1},S_i;\boldsymbol{\Phi}_{\mathbf{x}_i}), i=1,\dots,N$ , with more accurate distributed flow routing models, seems challenging; currently, no methodology allowing for modelling hazard only in the area between two hierarchy levels seems readily available literature. As such, when the accuracy of a distributed flow routing model is necessary, a different implementation approach is necessary. A possibility is to directly model hazard from source to receptor, with consideration of all protection structures and their respective states. That is, the flow routing model is run once for each hazard event  $hz_m, m=1,\dots,M$  and combination of states  $S_i, i=1,\dots,N$  (and, obviously, for each considered decision alternative in  $A_s$ ). Although it is easier to implement, challenges may arise from the sheer number of combinations of states  $S_i, i=1,\dots,N$ , which generally grows exponentially with the number of hierarchy levels. Approaches are available to model hazard in function of protection structure states (Dawson *et al.* 2005); however, they only consider one protection structure/hierarchy level, and must be adapted and tested for the consideration of more hierarchy levels.

A third challenge is fragility or vulnerability modelling. For all protection structures considered in the decision analysis, models allowing for the accommodation of decision alternatives are necessary. Dike fragility models are available, which are based on structural reliability theory and allow structural optimisation for flood risk management.

Not much research is found on fragility of local flood barriers. Whereas several vulnerability models for residential buildings are available that consider effects of flood proofing measures on building vulnerability, few are based on engineering models and none provide necessary modelling details to allow an immediate practical application.

Last, a fourth challenge is the comprehensive modelling of consequences. For a meaningful decision analysis, all types of consequences need to be considered, including indirect and intangible consequences. This is particularly important when considering a hierarchical flood protection system, which, when compared to single-structure protection systems, may imply higher probability of flooding for certain areas with low receptor value density. This can only be fully justified if all consequences are considered. As mentioned in Chapters 2 and 3, the comprehensive assessment of consequences, and in particular of indirect and intangible consequences, is challenging and might require further research.

#### *4.5.1 Challenges approached in this thesis*

This thesis makes a contribution towards the modelling of hierarchical flood protection systems. From the identified challenges, two are here selected and approached. The goals listed in the introduction are reiterated in the following.

First, a contribution towards consistent modelling of risk at different spatial scales is made. A probabilistic disaggregation model is developed allowing for disaggregating spatially aggregated portfolios of receptors with consideration of uncertainty and spatial correlation in the receptor distribution. The proposed model is documented in a paper submitted for publication to *Georisk* and is reproduced in Chapter 5.

Second, a vulnerability model for residential buildings is developed, allowing for representing the impact of flood proofing measures on vulnerability. The model can therefore be utilised as part of a risk-based decision analysis for flood proofing of buildings. The developed model is documented in a paper manuscript accepted for publication by *Natural Hazards* and is reproduced in Chapter 6.

To further illustrate the concept and implementation of the developed methodologies for flood risk analysis and for the modelling of vulnerability of residential buildings, they are applied to an example study area in Switzerland; this is described in Chapter 7.



## **5 Probabilistic disaggregation model (Paper 1)**

The present chapter describes a methodology for the probabilistic disaggregation of spatially aggregated amounts. The methodology can be used to disaggregate portfolios of receptors. The chapter is a reproduction of a paper manuscript under review at *Georisk*. As such, definition of variables and parameters, as well as terminology may be different in this chapter and the rest of the thesis.

### **The probabilistic disaggregation model within the thesis**

Probabilistic disaggregation may be necessary in flood risk assessment when models and data are not available at the same spatial resolution, and in particular, when data on the portfolio of receptors is available in a spatially aggregated form, while hazard is modelled at a higher spatial resolution. In such a situation, disaggregation is a possible method to transfer spatially aggregated data to a higher resolution.

The author acknowledges that in developed countries, the location and characteristics of portfolios of receptors is generally known with good precision by authorities. However, this data may only be available to the public in aggregated form for reasons of data protection.

For instance, insurance and reinsurance industry portfolios of receptors are often only available in aggregated form, and industry standards have long been established for the spatial aggregation of portfolio data (CRESTA 2015).

This issue of data and model resolution mismatch is not particular to hierarchical flood risk management as it may be encountered in any flood risk assessment, however when optimizing a hierarchical flood protection system, the likelihood that data needs to be transferred from one resolution to another increases.

## **Probabilistic Disaggregation of a Spatial Portfolio of Exposure for Natural Hazard Risk Assessment**

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**ABSTRACT:** In natural hazard risk assessment, situations are encountered where information on the portfolio of exposure is only available in spatially aggregated form, hindering a precise risk assessment. Recourse might be found in the spatial disaggregation of the portfolio of exposure to the resolution of the hazard model. Given the uncertainty inherent to any disaggregation, it is argued that the disaggregation should be performed probabilistically. In this paper a methodology for probabilistic disaggregation of spatially aggregated values is presented. The methodology is applied to the disaggregation of a portfolio of buildings in two communes in Switzerland and results are compared to sample observations. The relevance of probabilistic disaggregation uncertainty in natural hazard risk assessment is illustrated with the example on a simple flood risk assessment.

**Keywords:** disaggregation, portfolio, natural hazard, probabilistic, copula.

## 5.1 Introduction

Natural hazard risk models are widely utilised to assess the occurrence of natural hazard events such as floods, earthquakes and tropical cyclones, and their impact on the built environment and more generally on society. The level of sophistication of natural hazard risk models has been increasing over the past few decades and it has become possible to simulate hazard at high spatial resolutions. Practitioners in civil and environmental engineering, earth science, as well as risk modelling professionals in the insurance industry, are confronted with situations where hazard information is available at a higher spatial resolution relative to the information on the portfolio of exposures; that is, hindering a precise assessment of natural hazard risk. For the purpose of improving the quality of risk assessment the portfolio data has to be spatially disaggregated, i.e. mapped from a coarse to a finer resolution. The mapping can be performed according to indicators, where an indicator is auxiliary information which correlates with the disaggregated and is available at the finer resolution.

A common way to model disaggregation is to establish a statistical relationship between aggregated and disaggregated quantities, i.e. statistical disaggregation. Statistical disaggregation model can be either deterministic or probabilistic. In deterministic disaggregation, disaggregated quantities are a function of the indicator values with a given aggregated quantity. In probabilistic disaggregation, the disaggregated quantities are treated as random variables and thus consider uncertainty inherited in the process of disaggregation; i.e. disaggregation uncertainty.

In the context of portfolio disaggregation for natural hazard risk assessment, disaggregation uncertainty is due to lack of knowledge, i.e. is of epistemic nature. It only arises when a portfolio of exposure is aggregated and thus information on the spatial distribution at a higher resolution is not available. In any risk assessment situation it is preferable to acquire data of higher resolution on the spatial distribution of the portfolio of exposures. However, this is sometimes not possible because of economic or technical reasons, making disaggregation a necessary compromise. If the disaggregation uncertainty is neglected in a natural hazard risk assessment the tails of the risk distribution may be underestimated. This can lead to e.g. suboptimal decision making in the context of natural hazard risk management or to an underestimation of risk premium in an insurance context.

This paper aims at providing a methodology for probabilistic disaggregation of spatially aggregated quantities with a focus on the disaggregation of portfolios of

exposures for natural hazard risk assessment. Although the main focus of the paper is natural hazard risk assessment, the proposed methodology can be straightforwardly applied to similar spatial disaggregation problems in other applications.

### 5.1.1 Literature review

Few methods for portfolio disaggregation for natural hazard risk assessment are available in literature. Thieken *et al.* (2006) and Wünsch *et al.* (2009) give an overview on deterministic portfolio disaggregation methods and how the method choice affects flood risk assessment. Custer and Nishijima (2012) present a probabilistic disaggregation model based on the Dirichlet distribution.

More works can be found in the literature in other contexts, for example, disaggregation of precipitation. In the following, spatial disaggregation model for the disaggregation of precipitation are first reviewed. Thereafter, attention is turned to two challenges in disaggregation modelling, namely modelling of dependent non-Gaussian random fields as well as modelling of compositional data.

Precipitation time series disaggregation in the temporal dimension has been long utilised for the purpose to temporally interpolate rain gauge measurements. In recent years the spatial disaggregation of precipitation fields has also become increasingly important, e.g. when modelling local impacts of climate change. Future climate subject to climate change can be modelled with global circulation models (GCM) at resolutions above a few 10km and generally in the order of 100-200km (Chandler *et al.* 2000). In order to assess the impact of the climate change at a local scale, however, a finer resolution is required. Thus, the output of the GCM has to be down-scaled, or disaggregated.

Several probabilistic precipitation disaggregation models in contiguous space, i.e. on a lattice, are found in literature. A common approach is the multiplicative cascade technique, which takes its basis in spatial fractality of precipitation fields and is found in e.g. Over and Gupta (1996), Shrestha *et al.* (2005), Rupp *et al.* (2012). In these publications, disaggregation is achieved by constructing spatial precipitation fields from discrete multiplicative cascades of independently identically distributed generator variables. With the cascade technique the aggregated amount is disaggregated in several sequential steps. In the first step, the aggregated precipitation amount is disaggregated to very coarse grid cells. In each subsequent step each coarse grid cell is divided into smaller grid cells, and the precipitation amount assigned to the coarse grid cell is

distributed to the smaller grid cells. A spatial correlation structure is inherent to this procedure, since the step-wise disaggregation introduces dependence between neighbouring grid cells. However, an additional (and generally unwanted) correlation pattern inherent to the methodology is also introduced, as the resulting correlation is smaller across the cell borders of the most coarse grid cells. Attempts to overcome this inconvenience have been undertaken in e.g. Shrestha *et al.* (2005), where the modelling of spatial correlation is improved by conditioning disaggregated variables at a fine resolution on the neighbouring disaggregated variables at a more coarse resolution.

An assumption behind the multiplicative cascade approach is the presence of a fractal pattern in the disaggregated quantities; which, however, seems an improbable characteristic for portfolios of exposures.

Another approach takes basis in Markovian methods, where the random variable in each grid cell is conditional on neighbouring grid cells to create a Markov field. Examples of Markovian disaggregation models are found in Allcroft and Glasbey (2003), Chandler *et al.* (2000), Mackay *et al.* (2001), which employ image reconstruction techniques to downscale the output of a GCM to a local resolution. In general, grid cells are first identified as wet or dry according to a Markov random field and thereafter rainfall intensity is allocated to each wet grid cell as a random variable. The average rainfall intensity for a given grid cell is determined as a function of the inverse distance to the nearest dry cell, implicitly creating spatial correlation between neighbouring grid cells. Another Markovian approach is found in Gagnon *et al.* (2012), where precipitation is disaggregated with a Gibbs sampling algorithm. Spatial correlation of disaggregated variables is modelled through dependence between neighbouring grid cells and sequentially updated with the Gibbs sampling algorithm.

Non-Gaussian random fields are often modelled through transformation of Gaussian random fields. A flexible approach to modelling dependent non-Gaussian random fields is to represent the dependence structure with copula. In this approach marginal distributions and correlation structures, which together define the joint probability distribution, are separately modelled and brought together through the copula. A thorough introduction to copula from a risk management perspective is given in Embrechts *et al.* (2003). Copulae have been used in disaggregation methodologies, e.g. in the context of precipitation time series disaggregation, see Yang (2008).

Compositional data gives quantitative descriptions of parts of a whole. In a disaggregation problem, the aggregated quantity is the whole and disaggregated variables are parts of the whole and are therefore compositional variables. Seminal work

on the analysis of compositional data is found in Aitchison (1986). A prominent probability distribution to model compositional data is the Dirichlet distribution (Connor and Mosimann, 1969). Although it facilitates to formulate a variety of disaggregation problems, it does not provide much flexibility in the modelling of variance and correlation structure, since it implies a strictly negative correlation structure among variables. Several attempts have been made to render the Dirichlet distribution more general and flexible, see e.g. Connor and Mosimann (1969), Thomas and Jacob (2006), Ongaro *et al.* (2008), Ng *et al.* (2009)). However, the aforementioned shortcomings have yet to be completely overcome. Another popular approach to modelling compositional data is based on log-ratio transformations of compositional data, which is free from the problem of a constrained sample space and allows using standard multivariate techniques (Aitchison 1986).

### 5.1.2 Paper structure

In the present paper a new approach for probabilistic disaggregation onto contiguous lattice grid cells is proposed. The paper is structured as follows. The disaggregation methodology is first introduced followed by a description of the model characteristics. Thereafter, the model is applied to an example in the natural hazard risk assessment domain, and results are presented. Finally, the model performance is discussed and conclusions are provided.

## 5.2 Methodology

A methodology for probabilistic disaggregation to an array of  $N$  grid cells is introduced. It allows for probabilistically modelling spatial disaggregation of an aggregated variable  $X$  to disaggregated variables  $\mathbf{Z} = (Z_1, Z_2, \dots, Z_N)$ , according to indicators  $\mathbf{D} = (D_1, D_2, \dots, D_N)$ , with  $Z_i$  and  $D_i$ ,  $i = 1, 2, \dots, N$ , attributed to the  $i$ -th grid cell. Indicators  $D_i$  are considered categorical variables, which can take values  $d_i = \xi_j$ ,  $j = 1, 2, \dots, M$  and  $M$  is the number of different indicator values.

At first, the joint probability distribution function of  $\mathbf{Z}$  is expressed without considering the aggregated variable  $X$ , i.e. the unconditional joint probability distribution  $F_{\mathbf{Z}|\mathbf{D}}(z_1, z_2, \dots, z_n | \mathbf{d})$  is formulated. Disaggregated random variables  $Z_i$  are modelled marginally and their spatial correlations are considered through a Gaussian copula. Thereafter, variables  $Z_i$  are conditioned such that  $X = \sum_{i=1}^N Z_i$  to obtain the

conditional probability distribution  $F_{\mathbf{z}|\mathbf{x},\mathbf{D}}(z_1, z_2, \dots, z_N | x, \mathbf{d})$ . A closed form of the latter is generally not available, however, its characteristics are described further on.

### 5.2.1 Unconditional joint distribution

A probability distribution family to model  $Z_i$  should possibly be flexible and easy to parameterise and needs to consider that  $Z_i \geq 0$ . Possible distribution families are the gamma distribution and the lognormal distribution. The gamma distribution is chosen here to model marginal probability distribution of  $Z_i$ , because of its association with the Dirichlet distribution; namely, the Dirichlet distribution can be generated from a set of independent gamma distribution with a common rate parameter  $\beta$ . Therefore, in the proposed approach the probability density function of  $Z_i$  given indicator  $D_i$  is assumed to marginally follow the gamma distribution. A straightforward way to model the influence of the indicator is to express the distribution parameters as a function of the indicator, i.e. shape parameter  $\alpha_i = \alpha(d_i)$  and rate parameter  $\beta_i = \beta(d_i)$ . The marginal probability density function  $f_{z_i|D_i}(z_i | d_i)$  is then written as:

$$f_{z_i}(z_i | d_i) = f_{z_i}(z_i; \alpha(d_i), \beta(d_i)) = \frac{\beta_i^{\alpha_i}}{\Gamma(\alpha_i)} z_i^{\alpha_i - 1} e^{-\beta_i z_i}, z_i > 0, \quad (5.1)$$

where  $\Gamma(\cdot)$  is the complete gamma function.

The joint distribution function  $F_{\mathbf{z}|\mathbf{D}}(\mathbf{z} | \mathbf{d})$ , of  $\mathbf{Z}$  given indicators  $\mathbf{D}$  is modelled with the Gaussian copula (Embrechts *et al.* 2003), according to (5.2):

$$F_{\mathbf{z}|\mathbf{D}}(\mathbf{z} | \mathbf{d}) = \Phi_{N,\Sigma} \left( \Phi^{-1} \left( F_{z_1|D_1}(z_1 | d_1) \right), \Phi^{-1} \left( F_{z_2|D_2}(z_2 | d_2) \right), \dots, \Phi^{-1} \left( F_{z_n|D_n}(z_n | d_n) \right) \right). \quad (5.2)$$

$\Phi_{N,\Sigma}(\cdot)$  is the cumulative distribution function of the  $N$ -dimensional normal random variables with zero means and the correlation matrix  $\Sigma$ ,  $\Phi(\cdot)$  is the cumulative distribution function of the standard normal random variable,  $\Phi^{-1}(\cdot)$  is its inverse function and  $F_{z_i|D_i}(z_i | d_i)$  is the cumulative distribution function of  $Z_i$  given indicator  $d_i$ . The elements of the covariance matrix  $\Sigma$  are defined through a correlation function  $k(s; \phi)$ , where  $s$  is the distance between grid cell centroids and  $\phi$  is a correlation parameter, e.g. correlation length. A number of correlation functions can be found in e.g. Rasmussen and Williams (2006). In this paper an exponentially decaying correlation function is assumed, i.e.

$$k(s; \phi) = e^{-s/\phi}. \quad (5.3)$$

### 5.2.2 Conditional joint distribution

In disaggregation problems, the value of the aggregated variable  $X$  is known, and the variables  $Z_i$ ,  $i=1,2,\dots,N$ , are constrained by the equation  $\sum_{i=1}^N Z_i = x$ . To respect this constraint, variables  $Z_i$ ,  $i=1,2,\dots,N$  are conditioned on  $x$  as follows:

$$Z_i | x = \frac{Z_i}{\sum_{i=1}^N Z_i} \cdot x. \quad (5.4)$$

Evidently, Equation (5.4) ensures that the disaggregated variables sum up to the aggregated variable, while maintaining proportions between variables  $Z_i$  invariant. The conditional joint probability distribution  $F_{Z|X,D}(z_1, z_2, \dots, z_N | x, \mathbf{d})$  is defined through Equations (5.1), (5.2), (5.3) and (5.4).

### 5.2.3 Parameter estimation

Gamma distribution parameters and correlation parameters are generally estimated from sample observations, which are assumed to be available at the spatial resolution of disaggregated variables. It is further assumed that the samples are associated with a lattice of square grid cells and that spatial location at which each sample was collected is known.

Given a sample set, the parameter estimation is generally straightforward with standard estimation methodologies. For each indicator value  $\xi_j$  of interest, the shape parameter  $\alpha(\xi_j)$  and scale parameter  $\beta(\xi_j)$  are estimated e.g. using method of moment estimators,

$$\hat{\alpha}(\xi_j) = \frac{\bar{z}^2(\xi_j)}{v_z(\xi_j)} \quad \text{and} \quad \hat{\beta}(\xi_j) = \frac{v_z(\xi_j)}{\bar{z}(\xi_j)}, \quad (5.5)$$

where  $\bar{z}(\xi_j)$  is the sample mean and  $v_z(\xi_j)$  is the sample variance of the number of exposures per grid cell for indicator value  $\xi_j$ .

The estimation of the correlation length  $\phi$  in accordance with the proposed methodology requires transforming the observed sample data  $z_i$  to standard-normally

distributed sample data  $y_i$  using the estimated gamma distribution parameters  $\hat{\alpha}(d_i)$  and  $\hat{\beta}(d_i)$  as follows:

$$y_i = \Phi^{-1}\left(F_{Z_i}\left(z_i; \hat{\alpha}(d_i), \hat{\beta}(d_i)\right)\right), \quad (5.6)$$

where  $F_{Z_i}(\cdot)$  is the estimated gamma distribution function. The Pearson product-moment sample correlation coefficient  $\hat{\rho}(s)$  is calculated from all observed  $y_i$  for different values of distance  $s$ .

Given the choice of an exponential correlation function, the correlation parameter  $\phi$  can be estimated from the sample correlation coefficient  $\hat{\rho}(s)$  through appropriate regression methods.

### 5.3 Model characteristics

The model characteristics are described for two distinct cases. Firstly, a special case is analysed, where the closed form of the joint probability distribution function is known. Thereafter, the general case is characterised where the closed form of the distribution is not known.

#### 5.3.1 Special case with known joint probability distribution

The special case is given when the unconditional disaggregated variables  $Z_i$  are independent and parameter  $\beta_i$  is constant for all variables  $Z_i$ . The latter condition can be fulfilled either when the disaggregation problem only contains one indicator value or when  $\beta(\xi_j)$  is constant for all indicator values. When these conditions are satisfied, the disaggregation ratios  $W_i, i=1, \dots, N$ , defined for grid cell  $i$  as,

$$W_i = \frac{Z_i}{\sum_{i=1}^N Z_i}, \quad (5.7)$$

jointly follow the Dirichlet distribution and thus have probability density function

$$f_{\mathbf{w}}(\mathbf{w}; \boldsymbol{\alpha}) = \frac{\Gamma\left(\sum_{i=1}^N \alpha_i\right)}{\prod_{i=1}^N \Gamma(\alpha_i)} \prod_{i=1}^{N-1} w_i^{\alpha_i-1} \left(1 - \sum_{i=1}^{N-1} w_i\right)^{\alpha_N-1}, \quad (5.8)$$

where  $\mathbf{W} = (W_1, W_2, \dots, W_N)$  and  $\boldsymbol{\alpha} = (\alpha_1, \alpha_2, \dots, \alpha_N)$ .

The conditional joint probability density function  $f_{\mathbf{z}|x}(z|x; \boldsymbol{\alpha})$  is obtained from Equation (5.8) through variable transformation and follows a Dirichlet distribution with scaled support simplex given in Equation (5.9):

$$f_{\mathbf{z}|x}(\mathbf{z}|x; \boldsymbol{\alpha}) = \frac{\Gamma\left(\sum_{i=1}^N \alpha_i\right)}{\prod_{i=1}^N \Gamma(\alpha_i)} x^{\left(1 - \sum_{i=1}^N \alpha_i\right)} \prod_{i=1}^{N-1} z_i^{\alpha_i - 1} \left(x - \sum_{i=1}^{N-1} z_i\right)^{\alpha_N - 1}. \quad (5.9)$$

Marginally,  $Z_i|x$  follows a beta distribution with scaled support:

$$f_{Z_i|x}(z_i|x; \boldsymbol{\alpha}) = \frac{1}{B(\alpha_i, \alpha_+ - \alpha_i)} \frac{z_i^{\alpha_i - 1} (x - z_i)^{\alpha_+ - \alpha_i - 1}}{x^{\alpha_+ - 1}}, 0 \leq z_i \leq x, \quad (5.10)$$

where  $\alpha_+ = \sum_{i=1}^N \alpha_i$  and  $B(\cdot, \cdot)$  is the beta function.

The compositional constraint introduced in Equation (5.4) causes the covariance between two conditional variables  $Z_i$  and  $Z_j$ ,  $i, j = 1, 2, \dots, N$  to become negative with value

$$\text{Cov}[Z_i, Z_j|x] = -\frac{\alpha_i \alpha_j}{\alpha_+^2 (\alpha_+ + 1)}. \quad (5.11)$$

The Dirichlet distribution is closed under subcomposition and amalgamation (Monti *et al.* 2011). These are valuable characteristics when performing disaggregation, as it allows for performing sequential disaggregation (see Figure 5.3).

### 5.3.2 General case

For the general case with dependent variables  $Z_i$  and/or different values for parameter  $\beta_i$ , closed forms of the joint probability distribution functions  $f_{\mathbf{z}}(\mathbf{z}; \boldsymbol{\alpha}, \boldsymbol{\beta})$  and  $f_{\mathbf{z}|x}(\mathbf{z}|x; \boldsymbol{\alpha}, \boldsymbol{\beta})$  are not available. In the following, several characteristics of the latter are described in terms of moments of the marginal distribution. Moreover the correlation between disaggregated variables is characterised in terms of correlation coefficients as function of distance  $s$ .

### 5.3.2.1 Conditional marginal distribution

While the unconditional marginal distribution  $f_{Z_i}(z_i; \boldsymbol{\alpha}, \boldsymbol{\beta}), i=1,2,\dots,N$  are gamma distributed, the conditional marginal distributions  $f_{Z_i|x}(z_i|x; \boldsymbol{\alpha}, \boldsymbol{\beta})$  follow an unspecified probability distribution family. Their shape not only depend on  $f_{Z_i}(z_i; \alpha_i, \beta_i)$  but also on disaggregation problem size  $N$ , aggregated value  $x$ , indicators of all grid cells, correlation function  $k(s; \phi)$  and the correlation parameter  $\phi$ . In the following the expected value and variance of  $f_{Z_i|x}(z_i|x; \boldsymbol{\alpha}, \boldsymbol{\beta})$  are described analytically where possible, and with a numeric study otherwise.

The expected value  $E[Z_i|x]$  is determined by the unconditional expected value  $E[Z_i]$  and the normalisation in Equation (5.4), and can be expressed as:

$$E[Z_i|x] = \frac{E[Z_i]}{\sum_{i=1}^N E[Z_i]} \cdot x. \quad (5.12)$$

The value of  $Var[Z_i|x]$  depends on  $Var[Z_i]$  and parameters  $N$  and  $\phi$ . Since no analytical expression for  $Var[Z_i|x]$  is available, the influence of  $N$  and  $\phi$  on  $Var[Z_i|x]$  is analysed by a numeric study. Results are illustrated in Figure 5.1 in terms of the ratio  $Var[Z_i|x]/Var[Z_i]$ . In this numerical study variables  $Z_i, i=1,2,\dots,N$  are assumed to be identically distributed and that  $x = E\left[\sum_{i=1}^N Z_i\right]$ . As seen in from Figure 5.1, when  $Z_i$  are identically distributed,  $Var[Z_i|x] < Var[Z_i]$  for any value of  $N$  and  $\phi$  and the value of  $Var[Z_i|x]$  decreases for increasing value of  $\phi$  and decreasing value of  $N$ .

Note that for different value of  $x$ , the variance scales as follows:

$$Var[Z_i|x_1] = \left(\frac{x_1}{x_2}\right)^2 Var[Z_i|x_2]. \quad (5.13)$$

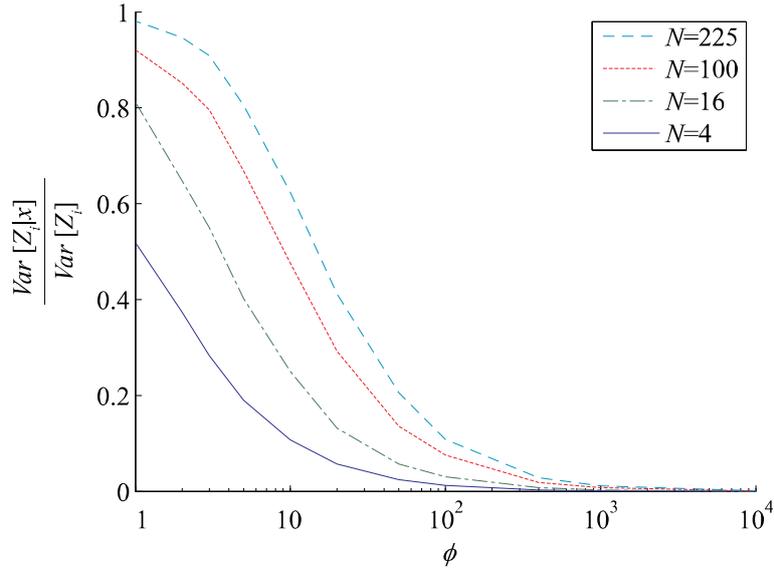


Figure 5.1 - Ratio  $\text{Var}[Z_i|x]/\text{Var}[Z_i]$  as a function of correlation length  $\phi$  for different values of  $N$

$$\text{and for } x = E\left[\sum_{i=1}^N Z_i\right]$$

For disaggregation problems where  $\text{Var}[Z_i|x] \cong \text{Var}[Z_i]$  (i.e. problems with small correlation length  $\phi$  and sufficiently large  $N$ ), the conditional marginal distributions  $f_{Z_i|x}(z_i|x; \mathbf{\alpha}, \mathbf{\beta})$  can be well approximated with the scaled beta distribution given in Equation (5.10).

### 5.3.2.2 Correlation structure

The correlation structure of conditional variables  $Z_i|x, i=1, \dots, N$  is determined by two contrasting effects. Firstly, the correlation structure of the unconditional variables  $Z_i, i=1, \dots, N$  defined by the correlation function  $k(s; \phi)$  and the Gaussian copula in Equation (5.2) generally specifies positive correlation. In contrast, the compositional constraint in Equation (5.4) introduces a negative correlation component, since disaggregated variables  $Z_i$  'compete' for portions of  $x$ , and the gain of  $Z_i$  will be the loss of  $Z_j, i \neq j$ . Figure 5.2 illustrates the correlation coefficients for unconditional disaggregated variables and conditional disaggregated variable for different size of disaggregation problem, i.e. for different values of  $N$ , as a function of the distance. While the disaggregation problem size  $N$  has no impact on the unconditional correlation structure, a smaller value of  $N$  has a larger impact on the correlation structure of the conditional distribution.

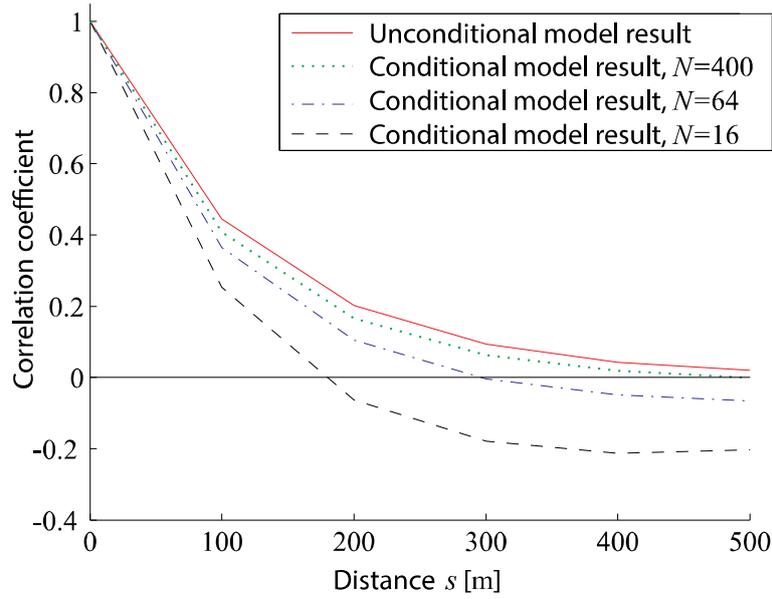


Figure 5.2 - Illustration of the difference in spatial correlation between unconditional and conditional variables

The correlation between neighbouring grid cells with different indicator value  $\xi_j$  is generally not modelled correctly. The Gaussian copula transfers correlation from Gaussian variables to gamma variables through matching of the respective quantiles. As a consequence, when correlated grid cell have different indicator and thus different marginal distributions, their quantiles correlate, however, not necessarily the realisation of the variable; hence, a bias.

A further bias in the correlation structure occurs when the model is used in a sequential disaggregation. The source of the bias is similar to the source of artificial correlation patterns produced by the multiplicative cascade disaggregation models, see the section for literature review. This is illustrated at the example of a two-step disaggregation in Figure 5.3. Disaggregated variables at a coarse resolution  $Z_k^*, k = 1, \dots, N_k$ , are introduced; in the first disaggregation step  $x$  is disaggregated to  $\mathbf{Z}^* = (Z_1^*, Z_2^*, \dots, Z_{N_k}^*)$ , i.e.  $f_{\mathbf{Z}^*|x}(\mathbf{z}^*|x)$  is modelled. In the second disaggregation each  $Z_k^*, k = 1, \dots, N_k$  is further disaggregated to  $Z_i, i = 1, \dots, N$ , i.e.  $f_{\mathbf{Z}|Z_k^*}(\mathbf{z}|z_k^*)$ ,  $k = 1, \dots, N_k$  is modelled.

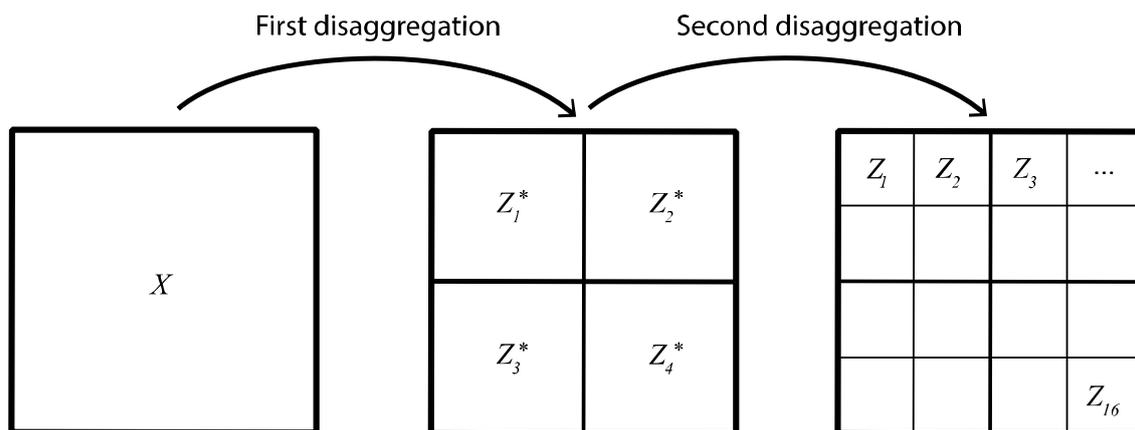


Figure 5.3 - Schematic illustration of a two-step sequential disaggregation

The presented methodology models spatial correlation of coarse resolution variables  $Z_k^*$ ,  $k = 1, \dots, N_k$  correctly, however, a systematic bias arises when modelling variables  $Z_i | Z_k^*$  because correlation between neighbouring variables at fine resolution is not considered across boundaries of the variables at coarse resolution. As a consequence, spatial correlation is slightly underestimated and takes an artificial pattern along the boundaries of coarse grid cells.

#### 5.4 Example application

The performance of the model based on the proposed methodology is illustrated with the example of a disaggregation of residential building portfolio of two communes, Burgdorf and Ittigen, in the Canton Bern, Switzerland. The CORINE land cover data is used for the indicator (EEA 2006); it provides land cover data for all European countries at a 100m resolution. For this application, only grid cells with CORINE land cover class 111 ( $\xi_1$ , urban) and 211-244 ( $\xi_2$ , agricultural) are considered. All other CORINE land cover classes, e.g. forests, industrial areas, water bodies, are either not present in the communes or considered uninhabited.

In the first step, sample observations of the number of buildings per grid cell are collected for each commune and indicator. The Burgdorf sample set is used as a training set, i.e. to estimate parameters, and the Ittigen sample set is used to cross validate the methodology. Here the underlying assumption is that both communes have the identical statistical characteristics up to the second central moment. Having estimated the parameters, disaggregation is modelled in both communes. The disaggregation results are compared to the respective sample statistics to validate the methodology. Finally the

example is extended with a simple flood risk assessment for Burgdorf, with the goal to illustrate the impact of probabilistic disaggregation on natural hazard risk assessment.

#### 5.4.1 Sample observations and statistics

Sample observations of the number of buildings per grid cell are collected with the use of satellite imagery; the CORINE land cover data as well as commune boundaries, are overlaid to satellite imagery of the communes and the number of buildings per grid cell is visually counted and classified according to the indicator value. In order to allow an estimation of spatial correlation, the location of each grid cell is also registered. Figure 5.4 illustrates the data utilised for the sampling for the commune of Ittigen.

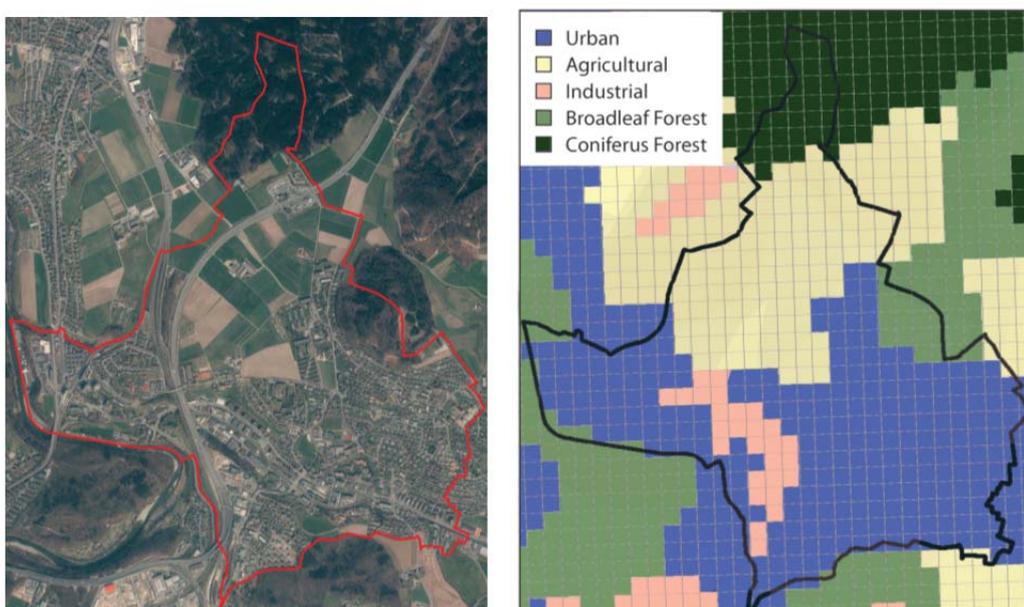


Figure 5.4 - Data for the commune of Ittigen. On the left side satellite imagery (Bing Maps) and commune boundary; on the right side, CORINE land cover data and commune boundary

The sample statistics for the sampled grid cells are given in Table 5.1; histograms of the sample sets are illustrated in Figure 5.5. Moreover, Figure 5.6 illustrates the sample correlation coefficients for urban grid cells in both communes as a function of the distance. Note that the correlations among agricultural grid cells, and between agricultural and urban grid cells, are not sampled.

The aggregated value, i.e. the total number of buildings per commune, is  $x = 2486$  for Burgdorf and  $x = 1446$  for Ittigen.

Table 5.1 - Sample statistics for each commune and indicator

Indicator	$\xi_1$ (Urban)		$\xi_2$ (Agricultural)	
	Burgdorf	Ittigen	Burgdorf	Ittigen
Commune				
Sample number	351	231	412	104
Sample mean $\bar{z}_i(d_i = \xi_j)$ [buildings / grid cell]	6.69	6.19	0.34	2.01
Sample variance $v_z(d_i = \xi_j)$ [buildings / grid cell] <sup>2</sup>	15.55	14.72	0.16	0.37

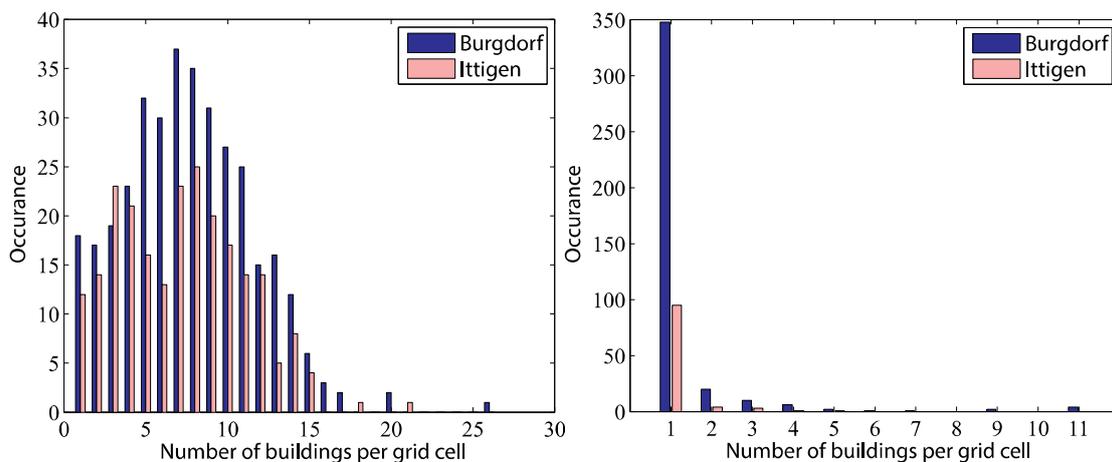


Figure 5.5 - Histograms of sample sets, on the left for urban, on the right for agricultural grid cells

Sample Correlation Coefficient

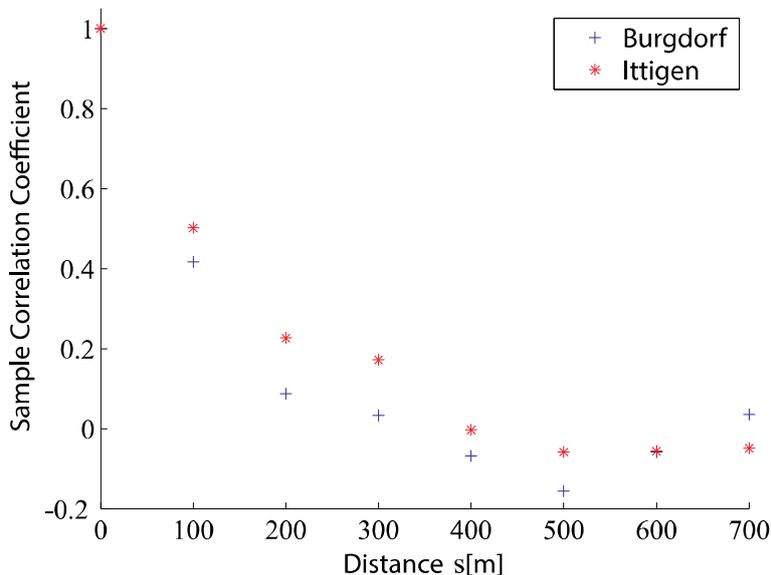


Figure 5.6 - Sample correlation coefficients as a function of distance  $s$

#### 5.4.2 Parameter estimation

Parameters are estimated separately for urban and agricultural areas from the training sample set, i.e. the Burgdorf sample set. The gamma distribution parameters, summarised in Table 5.2, are calculated by inserting the sample moments in Table 5.1 into the estimators in Equation (5.5).

Table 5.2 - Estimated Gamma distribution parameters by indicator and sample set

Indicator	Sample set	Parameter estimates	
		$\hat{\alpha}$	$\hat{\beta}$
$\xi_1$ (Urban)	Burgdorf	2.88	2.33
$\xi_2$ (Agricultural)	Burgdorf	0.056	5.97

To estimate the sample correlation parameter, the collected samples are first transformed to standard normal samples according to Equation (5.6), for which the gamma distribution parameters in Table 5.2 are utilised. Thereafter, the sample Pearson product-moment correlation coefficient of the standard normal samples is estimated for different distances  $s$ . Visually analysing the functional relation between correlation coefficient and distance, a suitable correlation function is identified in the exponential function in Equation (5.3) where both  $s$  and  $\phi$  are in meters. Parameter  $\phi$  is estimated by linearizing the correlation function  $k(s; \phi)$  followed by a regression analysis; the estimate  $\hat{\phi}$  is given in Table 5.3. Note that only urban grid cells are considered in this estimation, since the majority of buildings are found in urban areas.

Table 5.3 - Estimated correlation parameter

Indicator	Sample set	$\hat{\phi}$
$\xi_1$ (Urban)	Burgdorf	125m

Figure 5.7 illustrates the estimated correlation function together with the sample correlation coefficient  $\rho_{y_i}$  of the transformed normal samples with 95% confidence interval, which is calculated using the Fischer z-transform.

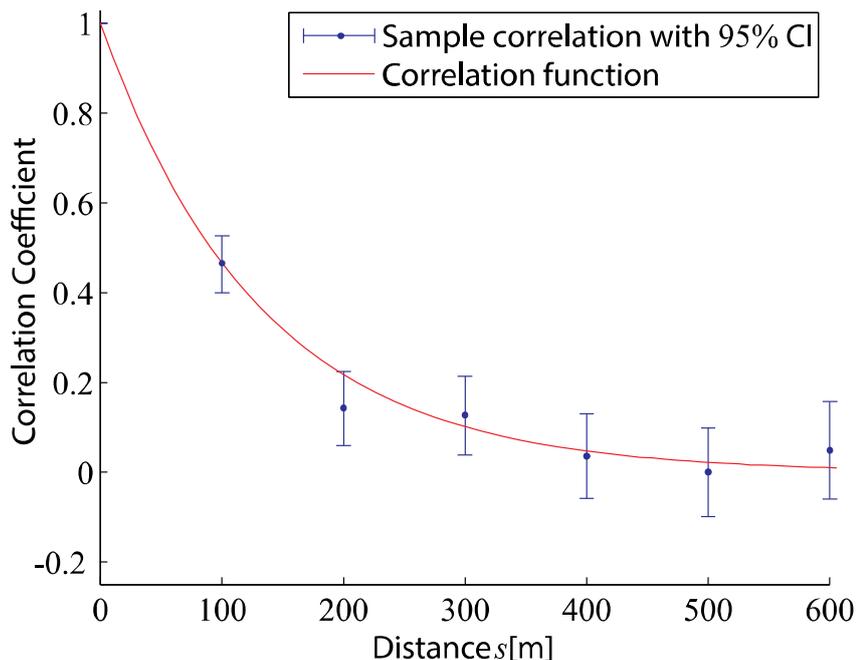


Figure 5.7 - Sample correlation coefficients with 95% confidence interval and the estimated correlation function

### 5.4.3 Disaggregation

Disaggregation is performed separately for each commune. As the parameters are estimated based on the Burgdorf samples, the Burgdorf disaggregation aims at validating the proposed methodology and the parameter estimates, whereas the Ittigen disaggregation aims at cross validating the methodology for the purpose of making predictions in study areas where no sample observations are available.

Each disaggregation is performed in the form of a Monte Carlo simulation with 100,000 realisations.

### 5.4.4 Flood risk assessment

The impact of the consideration of disaggregation uncertainty on natural hazard risk assessment is illustrated with a simple flood risk assessment for the commune of Burgdorf.

A set of 15 postulated flood hazard events are generated through the linear interpolation methodology, see Apel *et al.* (2009). The river Emme, which crosses Burgdorf, is assumed to be the source of flooding, and the events are defined by the river's water level at river entry point into the commune, see Figure 5.8. The water levels are chosen such that the smallest event barely causes an inundation, and in each further event the water level at entry point is increased by 0.5m. The water level at exit

is assumed to be 11m below the water level at entry. The annual exceedance probability for water level at the river entry into the commune is modelled with the Gumbel distribution with location parameter 532m.a.s.l. and shape parameter 1m.a.s.l; the resulting annual flooding probability is 25%.

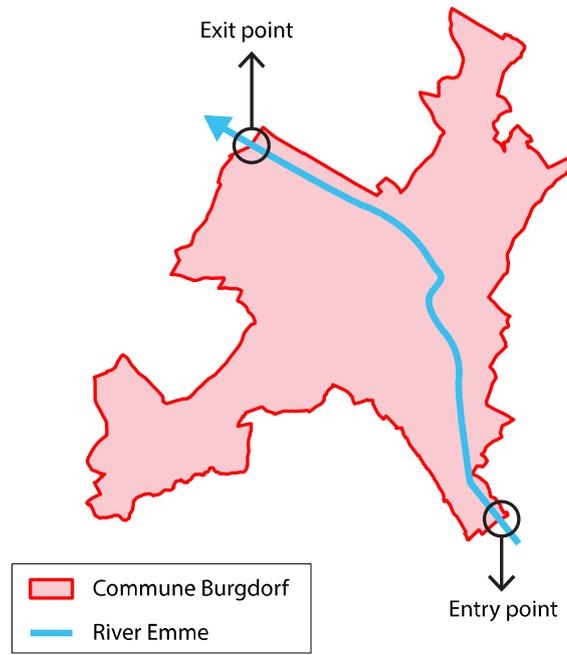


Figure 5.8 - Commune of Burgdorf with river Emme

Water level at any point in the commune is obtained through linear interpolation between entrance water level and exit water level. Water levels are intersected with the digital elevation model of the commune (Swisstopo 2005), allowing to identify which grid cells are inundated by an event. In Figure 5.9, three resulting inundation extents are illustrated for events 5, 10 and 15.

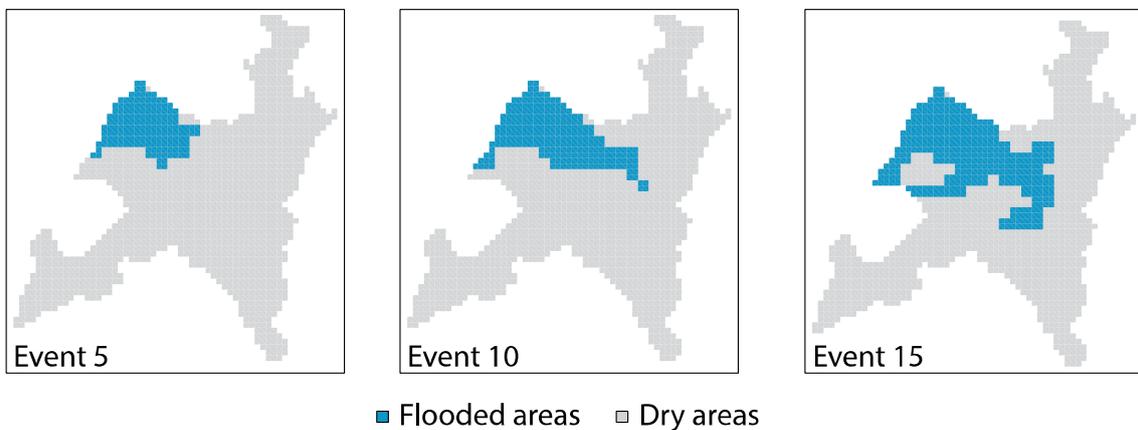


Figure 5.9 - Illustration of the inundation extent in three example events

The number of buildings affected by the flood is adopted as the proxy for quantifying the flood consequences. To assess the effect of considering disaggregation uncertainty on the risk distribution, a deterministic disaggregation model is required for comparison. In the comparison, the mode of the distribution of the number of affected buildings is taken as a representative deterministic value from the disaggregation model.

### 5.4.5 Results

In the following the results are presented. Modelling results are compared to sample statistics in terms of marginal distribution, as well as in terms of correlation coefficient in function of distance  $s$ . The results are thoroughly discussed in the following section.

#### 5.4.5.1 Marginal distributions

The observed sample distributions and modelled marginal distributions are compared by means of QQ-plots in Figure 5.10. for Burgdorf and Figure 5.11. for Ittigen. Moreover, in Table 5.4 the sample moments given in the previous sections are compared to the modelled distribution moments.

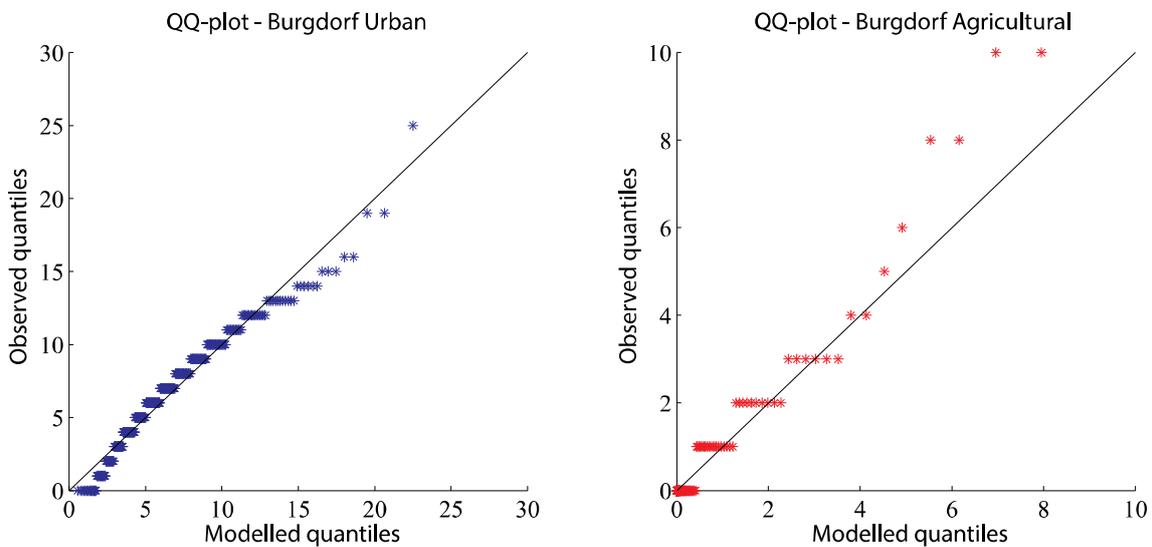


Figure 5.10 - QQ-plots for Burgdorf; on the left for urban grid cells, on the right for agricultural grid cells

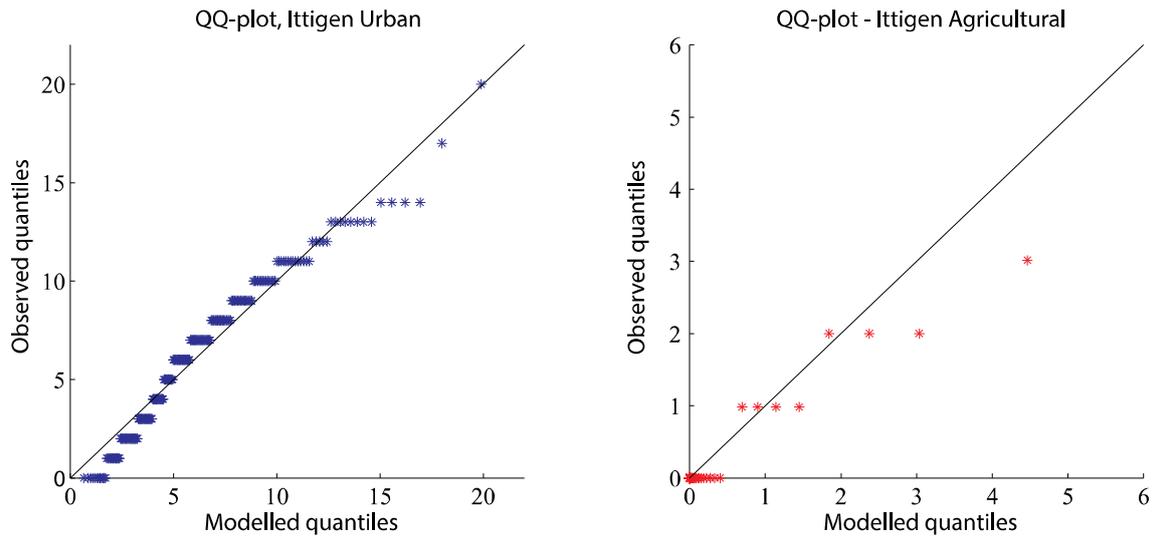


Figure 5.11 - QQ-plots for Ittigen; on the left for urban grid cells, on the right for agricultural grid cells

Table 5.4 - Comparison between sample moments and distribution moments

Indicator	$\xi_1$ (Urban)		$\xi_2$ (Agricultural)	
	Burgdorf	Ittigen	Burgdorf	Ittigen
$\bar{z}_i(d_i = \xi_j)$ [buildings / grid cell]	6.69	6.19	0.34	2.01
$E[Z_i   x, d_i = \xi_j]$ [buildings / grid cell]	6.69	6.13	0.335	0.31
$v_z(d_i = \xi_j)$ [buildings / grid cell] <sup>2</sup>	15.55	14.72	0,16	0.37
$Var[Z_i   x, d_i = \xi_j]$ [buildings / grid cell] <sup>2</sup>	15.29	12.70	1.97	1.66

#### 5.4.5.2 Spatial correlation

Disaggregation results are analysed in terms of spatial correlation and compared to sample correlation (Figure 5.12 and Figure 5.13). Whereas the correlation of both the unconditional and conditional distributions is presented, it is noted that the disaggregation result is given by the conditional distribution.

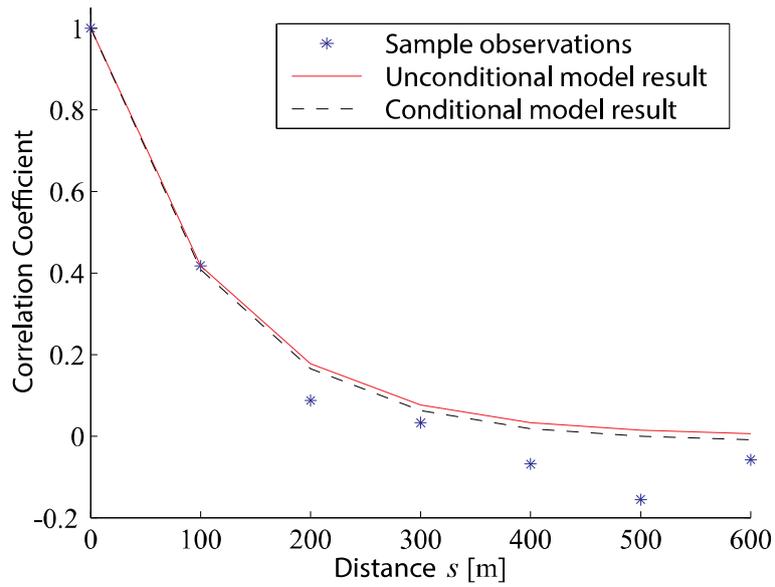


Figure 5.12 - Sample correlation coefficients and modelled correlation coefficients in function of grid cell centroid distance  $s$  for urban grid cells in the commune of Burgdorf

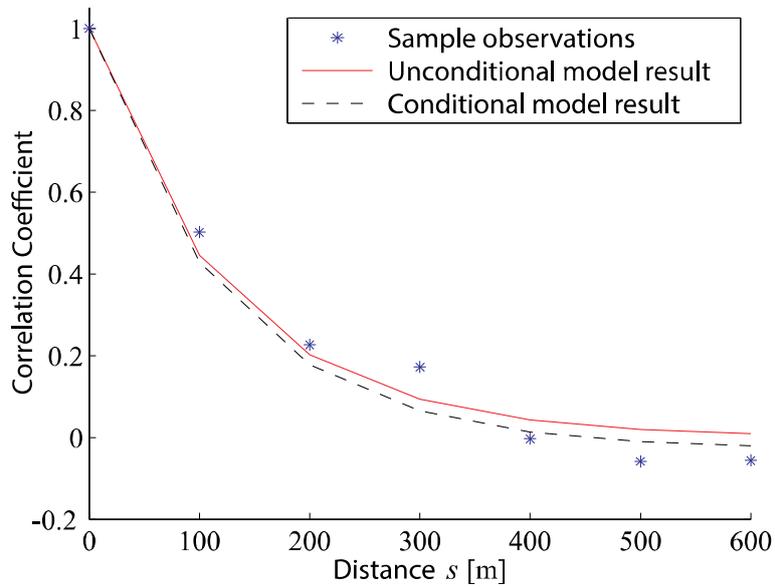


Figure 5.13 - Sample correlation coefficients and modelled correlation coefficients in function of grid cell centroid distance  $s$  for urban grid cells in the commune of Ittigen

5.4.6 Flood risk assessment

The results of the flood risk assessment are illustrated in Figure 5.14 and Figure 5.15. Figure 5.14 illustrates the number of affected buildings per event, comparing the deterministic result with the probabilistic result. Figure 5.15 illustrates the annual exceedance probability of the number of affected buildings for both models.

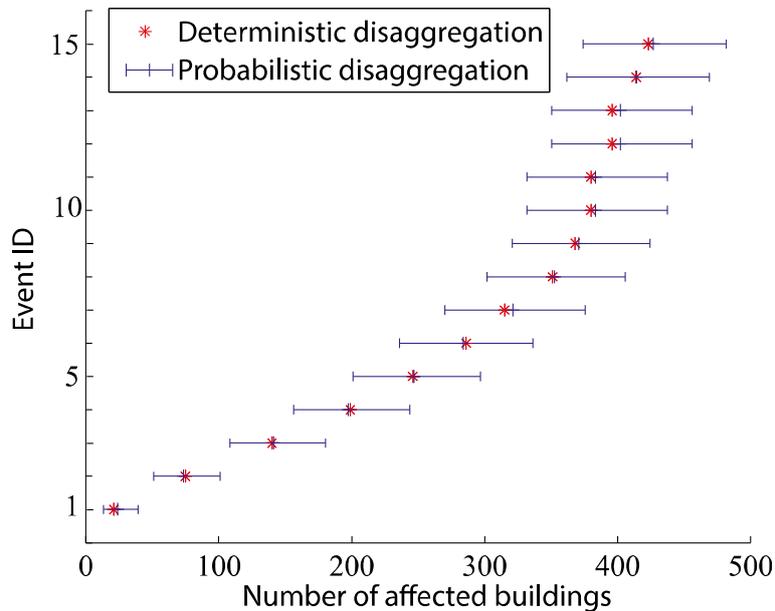


Figure 5.14 - Flood consequences per event in terms of number of affected buildings. The result from the probabilistic disaggregation is given by the distributions mean and the 5%-95% quantiles

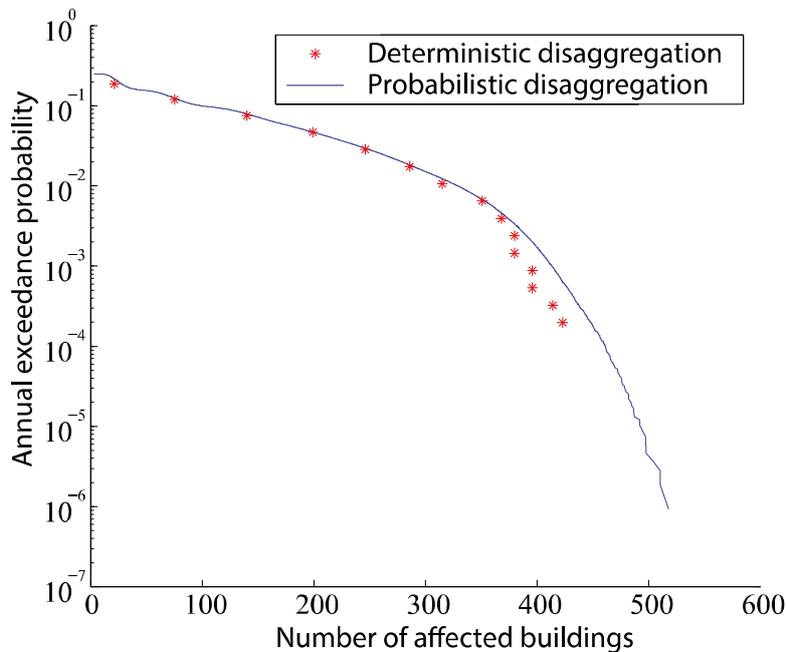


Figure 5.15 - Annual exceedance probability of the number of affected buildings

### 5.5 Discussion

The presented methodology allows for disaggregating a spatially aggregated variable under due consideration of disaggregation uncertainty and spatial correlation between variables. In the methodology, marginal distributions and correlation structure of disaggregated variables are modelled independently of each other and brought together by a Gaussian copula to a joint probability distribution function.

In the example application, the comparison between results and sample observations indicate that both marginal distribution and correlation are reproduced with a precision deemed satisfactory for most engineering applications, including natural hazard risk assessment, see Figure 5.10 to Figure 5.13.

As previously explained, it generally cannot be expected that observed sample distributions and conditional marginal distribution  $F_{Z_i|x}(z_i|x; \boldsymbol{\alpha}, \boldsymbol{\beta})$  match. Whereas the unconditional distribution  $F_{Z_i}(z_i; \boldsymbol{\alpha}, \boldsymbol{\beta})$  is modelled to follow sample observations and should therefore match,  $F_{Z_i|x}(z_i|x; \boldsymbol{\alpha}, \boldsymbol{\beta})$  is also influenced by the size of the disaggregation problem, the type of indicator of all other grid cells, the value of  $x$  and the correlation structure between variables  $Z_i, i = 1, \dots, N$ . However, for disaggregation problems of the size and parameterisation of the present example a reasonably good match between sample observation and conditional marginal distributions  $F_{Z_i|x}(z_i|x, \mathbf{d})$  is obtained.

The results for Burgdorf illustrated in Figure 5.10, Figure 5.12 and Table 5.4 compare well with sample observations, in particular for urban grid cells. This is not surprising as the parameters were estimated from the Burgdorf sample set. Nevertheless, it is noted that whereas the expected value for agricultural grid cells is well modelled, the variance is overestimated. This can be explained as follows: due to the compositional constraint, the  $Var[Z_i|x]$  is not only a function of parameters  $\alpha_i$  and  $\beta_i$  but also of all other disaggregated variables. Whereas  $E[Z_i|x]$  is modelled correctly for all variables following the conditioning in Equation (5.4),  $Var[Z_i|x]$  is predominantly determined by variables with large  $E[Z_i|x]$ , i.e. urban grid cells in this example, which cause the variance,  $Var[Z_i|x, d_i = \xi_2]$ , of agricultural grid cells is overestimated.

The results for Ittigen, given in Figure 5.11, Figure 5.13 and Table 5.4 compare less well with sample observations. Whereas the expected value  $E[Z_i|x, d_i = \xi_1]$  is well modelled, the variance  $Var[Z_i|x, d_i = \xi_1]$  is underestimated. For agricultural grid cells,

both expected value and variance compare poorly to sample observations. Two reasons to explain these deviations are given. Firstly, the cross validation can only be successful to the extent that the sample sets in the two commune are comparable, which was assumed in the example. In particular for agricultural grid cell the comparison is suboptimal. Secondly, the variance,  $Var[Z_i | x, d_i = \xi_2]$ , of agricultural grid cells is biased for the same reasons observed for Burgdorf.

The correlation functions seem to approximately fit the sample correlation coefficients in both communes. Several sample correlation coefficients, in particular in Burgdorf, are negative and can thus not be correctly modelled with an exponential correlation function.

As previously noted, the methodology cannot exactly represent correlation between variables associated with different indicator values. Correlations among agricultural grid cells, and between urban and agricultural grid cells, are not sampled nor specifically modelled. In the context of natural hazard risk assessment this is deemed acceptable and the bias of little impact on the risk distribution, because the majority of buildings is found in urban grid cells. However, in case where two or more indicators are differentiated for areas where buildings are dense, this bias may become significant.

The impact of disaggregation uncertainty on natural hazard loss distribution is exemplified with a simple flood risk assessment. Figure 5.14 illustrates the increase in variability of the flood consequences when disaggregation uncertainty is taken into account. In Figure 5.15, the results of all events are aggregated to risk curves. Comparing the risk curve obtained through deterministic disaggregation and probabilistic disaggregation, it can be observed that disaggregation uncertainty increases the variability of the loss distribution, i.e. it has fatter tails. This observation highlights the relevance of probabilistic disaggregation to accurately assess the tail risks in natural hazard risk assessment.

In future research it is recommended to apply the presented methodology to practical examples in both, risk management and insurance contexts, for a quantification of the effect of disaggregation uncertainty on risk distributions.

Several modelling aspects are simplified and should be improved in further research. For instance, it is not considered that building counts are better modelled with discrete variables rather than the continuous gamma distribution. In practice, the size of the disaggregation problem  $N$  is limited by the computation of the correlation matrix  $\Sigma$  for the Gaussian copula, which has  $N^2$  elements; when the  $\Sigma$  becomes prohibitively

large disaggregation problems can be approached through sequential disaggregation, at the cost of introducing an artificial spatial correlation pattern. Whereas the proposed methodology is applied to one example application, applications to other modelling context and with different parameterisations would be beneficial to further understand the methodologies properties and limitations.

This paper focuses on practical aspects of probabilistic disaggregation and the analytical formulation of closed form expression of joint probability distribution functions are not examined in depth, which is recommended further research.

### **5.6 Summary and conclusion**

The natural hazard risk assessment of portfolios of exposures often requires the disaggregation of spatially aggregated portfolios. Currently, disaggregation is often modelled deterministically, which can cause loss distribution tails to be underestimated. A sound approach is to model disaggregation in a probabilistic manner.

In the proposed probabilistic disaggregation methodology, disaggregated variables are marginally modelled with the gamma distribution and the spatial correlation structure is modelled with a Gaussian copula. The compositional constraint is modelled through a normalisation of the variables. The model characteristics are described for two cases: For the special case without spatial correlation and with constant parameter  $\beta_i, i=1,2,\dots,N$ , where the conditional joint probability distribution  $F_{\mathbf{z}|x}(\mathbf{z}|x; \boldsymbol{\alpha}, \boldsymbol{\beta})$  is known to follow a scaled Dirichlet distribution; for other cases a closed form expression is not available and the characteristics of the model are therefore described by means of numerical investigations.

In an example application, the model is applied to a portfolio disaggregation for two communes in Switzerland, the communes of Burgdorf, used as a training set to estimate parameters, and the commune of Ittigen, used for cross validation. The disaggregation results generally compare favourably with sample observations. The example application includes a simple flood risk assessment for the commune of Burgdorf, which facilitates to illustrate the importance of considering disaggregation uncertainty in natural hazard risk assessment.

The presented methodology contributes to natural hazard risk assessment by providing a simple and straightforward way of modelling disaggregation uncertainty, which in turns allows a more accurate natural hazard risk assessment.

## **6 Flood vulnerability assessment of residential buildings (Paper 2)**

The present chapter, a reproduction of a paper manuscript published by the journal *Natural Hazards*, describes a vulnerability modelling approach for residential building in floods. As such, definition of variables and parameters, as well as terminology may be different in this chapter and the rest of the thesis.

**Flood vulnerability assessment of residential buildings by explicit damage process modelling**

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**ABSTRACT:** The present paper introduces a vulnerability modelling approach for residential buildings in flood. The modelling approach explicitly considers relevant damage processes, i.e. water infiltration into the building, mechanical failure of components in the building envelope and damage from water contact. Damage processes are modelled at a building component level, utilizing engineering models where possible. The modelling approach is presented in general terms, which should be applicable to a large variety of building types. The paper illustrates the implementation of the approach for a 2-storey masonry building. Results are presented in terms of a parameter study for several building parameters and hazard characteristics, as well as, in terms of a comparison with damage data and literature vulnerability models. The parameter study indicates that hazard characteristics and building characteristics impact damage ratios as expected. Furthermore, the results are comparable to vulnerability models in literature. Strengths and shortcomings of the model are discussed. The modelling approach is considered as a step towards the establishment of vulnerability models that can serve as a basis for engineering decision making for flood risk management for residential buildings.

**Keywords:** Flood; Vulnerability; Residential building; Damage process modelling; Engineering vulnerability model.

## 6.1 Introduction

Flood vulnerability assessment of residential buildings aims at determining the direct consequences incurred by residential buildings in a flood event. Flood vulnerability assessment is therefore an integral part of any flood risk assessment, and is generally necessary for flood risk management.

Approaches to assess flood vulnerability of residential buildings have been available for several decades (see Smith 1994 for historical references). However, in recent years flood risk management has changed significantly, and hence, the requirements to vulnerability models. In particular, flood risk management has extended its focus from the construction of single large protection structures (e.g. dikes) towards an integrated approach, where flood risk is managed with a portfolio of structural and non-structural risk management measures (Jha *et al.* 2012). Amongst these, flood proofing measures for residential buildings (i.e. modifications to a building to reduce its vulnerability to floods, see e.g. Elliott and Leggett 2002) have gained increased attention, e.g. in Joseph (2014). When evaluating flood proofing measures, a cost-benefit analysis can indicate which flood proofing measure is most efficient. To consider flood proofing measures in a formal cost-benefit analysis, vulnerability modelling approaches are necessary, which fulfil the following requirements:

- Vulnerability needs to be modelled for individual buildings rather than in aggregated manner.
- Building and hazard characteristics need to be considered in detailed manner with engineering parameters.
- The vulnerability model needs to reflect the change in vulnerability from prospective flood proofing measures.

Furthermore, the modelling approach should be formulated in a general manner to allow its application to a large variety of building types.

Recent reviews of vulnerability models available for residential buildings in floods are found in Jongman *et al.* (2012) and Merz, Kreibich *et al.* (2010). The most common form of vulnerability model is a stage-damage curve, i.e. a function relating flood water depth to damages. Depending on the model, damages are represented in absolute monetary terms (absolute damage curve, see e.g. Penning-Rowsell *et al.* 2010), or as a ratio of building value (relative damage curve, see e.g. Scawthorn, Blais *et al.* 2006). Whereas the former is generally easier to identify, the latter has the advantage of better transferability in time and space, see Merz, Kreibich *et al.* (2010). Models also differ in

the way they are established, see e.g. Smith (1994). Empirical models are based on damage data from past events; the stage-damage curve is generally established through regression or comparable methodology. In contrast, synthetic models are established through what-if analyses on hypothetical buildings and expert judgment. Advantages and disadvantages of both approaches are summarised in Merz, Kreibich *et al.* (2010). Another key difference between models is the spatial aggregation level of modelling (Messner *et al.* 2007). Macro-scale models (e.g. Scawthorn, Blais *et al.* 2006) assess damages in aggregated manner at administrative unit scale. Meso-scale models (e.g. Thielen *et al.* 2008) generally take land-use into consideration and have resolution of 100m-1km. Finally, micro-scale models consider damages for individual objects at risk, e.g. buildings.

Empirical damage functions seem to be the most common type of vulnerability model. However, empirical vulnerability models generally do not provide the accuracy required to capture a reduction in vulnerability introduced by prospective flood proofing measures. An exception might be found in the recent development of an empirical vulnerability model based on Bayesian probabilistic networks, see Vogel *et al.* (2012), where a large number of explanatory variables are considered. Nevertheless, lacking large sets of data in good quality, a synthetic vulnerability model at micro-scale seems the best approach to address the outlined modelling requirements.

Several such models are available in literature, maybe most prominently the Multi-Coloured Handbook (Penning-Rowsell *et al.* 2010). Building upon seminal work from the 1970s by the same authors (e.g. Penning-Rowsell and Chatterton 1977), it details stage-damage curves for several residential building types in the UK in terms of absolute monetary losses. Losses are specified for individual building components and contents, and then summed up to building losses. The model considers water depth and flood duration as hazard parameters. Although the modelling approach is generally valid, the transferability of the stage-damage curves in time and space is limited, since the damages are specified in absolute monetary terms and because the modelled buildings are proprietary to the UK.

A further sophistication is found in synthetic vulnerability models, which explicitly consider relevant damage processes. By modelling damage processes, they are best indicated to be applicable to different building types, as it is reasonable to assume that physical processes that lead to damages are comparable between building types.

In Kelman (2002) a vulnerability analysis of residential buildings in the UK is detailed, which takes basis in the physical damage processes. Damages are modelled at

a component level, utilising engineering models where possible. The mechanical failure of wall and windows is modelled in detail. The work also emphasises the importance of considering the flood rise process and water infiltration process, as well as the water head differential between interior and exterior of the building. The model output is given in terms of discrete building damage state for combinations of water depth and water flow velocity. However, lacking a precise quantification of damages, the results of the vulnerability cannot be straightforwardly used for a cost-benefit analysis in regard to flood proofing of residential buildings. A similar model is found in Nadal *et al.* (2010); it provides stage-damage curves, albeit without much modelling details. De Risi *et al.* (2013) simulate damage processes and derive analytical vulnerability curves from the simulation results. In Mazzorana *et al.* (2014) relevant physical processes are modelled in great detail and under consideration of a multitude of system characteristics. The model allows for a cost-benefit analysis of flood proofing measures. Nevertheless, its application is limited to modelling of single buildings due to high computational demands.

Taking basis in damage process-based flood vulnerability models available in the literature, the present paper introduces a vulnerability modelling approach for residential buildings in floods. It allows for explicitly considering relevant hazard characteristics and buildings characteristics, and can reflect changes in vulnerability from implemented flood proofing measures. As such, it can be used as a base for cost-benefit analysis for flood proofing of residential buildings.

### *6.1.1 Characterisation of flood damage processes*

Before the proposed modelling approach is described in the next chapter, in the following a survey of literature on relevant flood and building characteristics and flood damage processes is given.

Building characteristics with the largest influence on flood damage include building type, building material and whether flood proofing measures are in place (Kreibich *et al.* 2005). Building type indicators can include information on the occupancy of the building (i.e. residential, commercial or industrial), on the construction type, (e.g. detached house, row house, terrace house, apartment house, see Penning-Rowsell *et al.* 2010), and the number of stories (USACE 1992). The role of building material is emphasised in Maiwald and Schwarz (2011), where a categorisation of buildings according to their vulnerability is proposed. The building material is relevant in two

regards: firstly, it is decisive in determining how fast water infiltrates the building; secondly, when the building is subject to large hydrostatic and hydrodynamic pressure, structural damage may occur depending on the building material (Kelman 2002). Next to the main building characteristics, relatively minor building details play a significant role in determining vulnerability (Wingfield *et al.* 2005). For instance, the presence or absence of a basement, window or an airbrick can determine whether building interiors get flooded or not.

Damages to buildings follow from the flood actions on the building, which are defined in Kelman and Spence (2004) as “...acts which a flood could directly do to a building, potentially causing damage or failure.”, and they may include “... energy transfers, forces, pressures, or the consequences of water or contaminant contact”. The same paper lists relevant flood actions: hydrostatic lateral pressure, hydrodynamic lateral pressure from water flow velocity, buoyancy forces, erosion, wave action, debris impact, and chemical and physical deterioration of building components due to water contact. Flood actions on a building follow from flood characteristics in proximity of the building, i.e. water depth, water flow velocity, flood rise rate and flood duration, debris flow and water contamination. According to the same paper, water depth is the flood characteristic with the largest correlation with damages, followed by water flow velocity, event duration and water contamination. Moreover, large emphasis is given to the flood rise rate and the water head differential, i.e. the difference between water depth inside and outside a building. According to an expert survey in Proverbs and Soetanto (2004), flood water depth and contaminant content are the most relevant flood characteristics for damage estimation. According to Penning-Rowsell (1981) water flow velocity is only relevant in rare cases of structural failure. Kreibich *et al.* (2009) confirms that water flow velocity shows limited correlation with building damage, and indicates that derived criteria combining water depth and water flow velocity perform better in explaining damages. Such criteria can be found in e.g. Clausen (1989) and USACE (1998). In Thieken *et al.* (2005) damage data is analysed as a function of flood event duration, indicating that extended flood durations tend to increase damages. The importance of a detailed consideration of hazard in analytical vulnerability studies is emphasised in Mazzorana *et al.* (2014).

From the multitude of flood actions a variety of damage mechanisms arise; an overview is given in Kelman and Spence (2003a). Most important damage mechanisms are: physical and chemical deterioration through water contact, failure of an external

wall or windows from lateral hydrostatic and hydrodynamic pressure, contamination, scour of foundation and buoyancy.

Damage to building components from water contact can arise through a number of different chemical or physical processes, e.g. metal corrosion, wood rotting, etc. Due to the multitude of damage processes, no general modelling approach taking basis in damage processes seems available. Recurs might be found in literature specific to the contact between different component categories and water (e.g. woods components, appliances, soft furnishing, electrical system etc.), which however, is not approached this paper. Büchele *et al.* (2006) proposes various types of parametric vulnerability curves which can be utilised to model damage from water contact.

Wall failure from lateral hydrostatic and hydrodynamic pressure is modelled in Kelman (2002) and Kelman and Spence (2003b) for masonry buildings, Nadal *et al.* (2010) for concrete frame buildings, and Roos (2003) for masonry and concrete buildings; the latter also considers debris impact. Further literature on the failure of walls under out-of-plane action is available, e.g. experimental results for masonry walls in USACE (1988). Window and door failure under lateral hydrostatic and hydrodynamic pressure is treated in Kelman and Spence (2003a). Modelling of foundation scour is described in Nadal *et al.* (2010). Recent literature on buoyancy of buildings in flood seems sparse; Black (1975) writes about buoyancy of rural buildings.

### 6.1.2 Goal and outline of this paper

In accordance with the formulated modelling requirements in the previous section, the proposed modelling approach aims at being applicable to a broad range of building types, however at the same time allow for detailed consideration of building and hazard characteristics.

The balancing between broad applicability and detailed consideration of system characteristics determines several modelling choices. Firstly, as in Penning-Rowse *et al.* (2010) and Kelman (2002), the building is considered a collection of components and damages are modelled at a building component level. Secondly, similarly to Kelman (2002) and Mazzorana *et al.* (2014), damages are assessed by explicit modelling of relevant damage processes, namely water infiltration process, damage from water contact and mechanical failure of components in the building envelope. Thirdly, wherever possible, engineering models are employed in the assessment.

The uniqueness of the presented work lays in the formulation of a modelling approach, which is general in its logic, consistently considers uncertainty and is based on the damage-process modelling at a building component level. The consideration of building components and damage processes is abstracted to make the modelling approach applicable to a large variety of buildings. In its implementation, however, the modelling approach allows for detailed consideration of characteristics of individual components and their behaviour, thus allowing for discriminating different realisations of building types. Furthermore, the proposed modelling approach allows for consistent treatment of uncertainty and uncertainty propagation and can be utilised to model individual buildings as well as portfolios of buildings.

In the next chapter the modelling approach is introduced. Thereafter, an example implementation of the modelling approach is described for a 2-storey masonry building. Results are presented in a parameter study, as well as with comparison to empirical damage data and vulnerability models in the literature. The paper concludes with a thorough discussion and an outlook on further research work.

## 6.2 Flood vulnerability modelling approach for residential buildings

In a general risk assessment modelling approach, adapted from JCSS (2008), the damaging hazard event  $HZ$ , with probabilistic characterisation  $P[HZ]$ , causes the considered system to be in damage state  $D$ , whose probabilistic characterisation is  $P[D|HZ]$ . In general, damage state  $D$  has both direct and indirect consequences.

However, in the present paper, only direct consequences are considered in terms of monetary loss  $L$ , modelled with the probability term  $P[L|D]$ , see Figure 6.1.

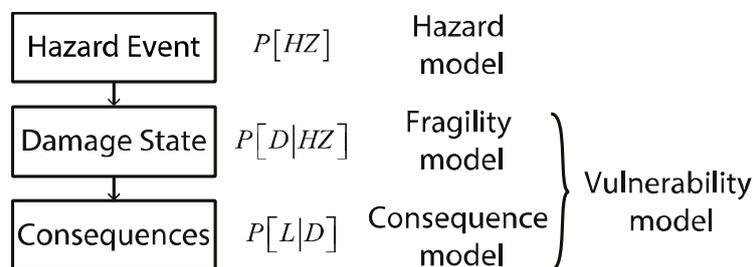


Figure 6.1 - General risk assessment framework, adapted from JCSS (2008)

The aim of the vulnerability model is to determine direct consequences  $L$  for given hazard event  $HZ$  under due consideration of uncertainties, that is:

$$P[L|HZ] = \int_{D_D} P[L|D]P[D|HZ]dD. \quad (6.1)$$

where  $D_D$  is the domain of  $D$ .

The present paper proposes a modelling approach for a vulnerability model, as defined in the figure and equation above, for residential buildings in flood.

Note that the paper makes an extensive use of conditional probability terms and the notation  $P[A|B]$  is used for different meanings. Whereas the notation is formally correct if  $A$  is an event. In case  $A$  is a random variable the notation implies the abbreviation of  $P[A=a|B]$ , i.e. the probability density (continuous case) or probability mass (discrete case).

In the following the representation of buildings and hazard events is introduced before considerations are made on damage processes and modelling thereof. A list of abbreviations introduced throughout this chapter is provided in Table 6.9 in the Annex of Chapter 6.

### 6.2.1 System representation

The considered system is a residential building. The behaviour of residential buildings in flood is complex and for given flood event  $HZ$  damage can vary greatly. Part of the complexity stems from the large number of building characteristics that influence vulnerability. This has to be considered in the building representation scheme in consistent and tractable manner. Additionally, the chosen system representation should facilitate an efficient analysis of probabilities and risk associated with a flood event. A hierarchical system representation, see e.g. Nishijima *et al.* (2009), seems an ideal choice. The building characteristics are primarily represented in terms of a hierarchical aggregation of the characteristics and interrelations of its components, i.e. the building is considered a collection of its components, see Figure 6.2.

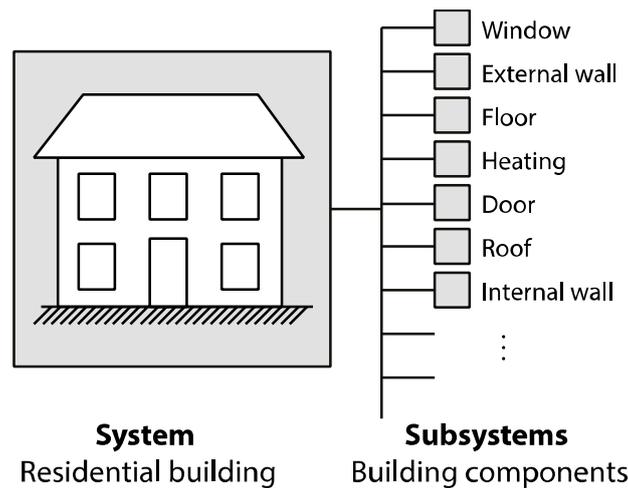


Figure 6.2 - Hierarchical representation of a building as a collection of components

The hierarchical representation has several advantages. First, due to their complexity, modelling building fragility by means of engineering models is challenging. Conversely, the behaviour in flood of individual components is generally reasonably well understood and can often be represented with an engineering model, i.e. it is easier to model component fragilities rather than building fragilities. Second, by modelling the building as a collection of components, changes to building configuration can easily be represented by adding or modifying individual components. Similar building representations are adopted in other flood vulnerability modelling approaches, e.g. Kelman (2002), Nadal *et al.* (2010) and Penning-Rowsell *et al.* (2010).

#### 6.2.1.1 Building parameterisation

Building characteristics that are not inherent to a component, such as the number of stories and the presence/absence of a basement are captured with building parameter vector  $\boldsymbol{\psi} = (\psi_1, \psi_2, \dots, \psi_{N_b})$ , where  $\psi_i, i = 1, 2, \dots, N_b$ , are building parameters and  $N_b$  is the number of building parameters. According to the hierarchical representation, all other characteristics of the building are captured at a component level. The building is considered a collection of  $N$  components  $C_j, j = 1, \dots, N$ , and each component  $C_j$  is characterised through a component parameter vector  $\boldsymbol{\gamma}_j = (\gamma_1, \gamma_2, \dots, \gamma_{N_j})$ , where  $\gamma_i, i = 1, 2, \dots, N_j$  are component parameters and  $N_j$  is the number of parameters needed to characterise  $C_j$ . Parameters in  $\boldsymbol{\gamma}_j$  characterise the monetary value of components, position within the building, functionality within the building and fragility in regard to all considered damage mechanisms. The elements in  $\boldsymbol{\gamma}_j$  may vary depending on the nature of the component. The collection of component parameter vectors of all

components is  $S_\gamma = \{\gamma_1, \gamma_2, \dots, \gamma_N\}$ .

### 6.2.1.2 Types of components

In the context of flood vulnerability modelling two component types are identified, which take a special role within buildings, namely structural and envelope components.

Structural components differ from other components in the sense that their failure can lead to the failure of the whole building.

Envelope components together form the building envelope, which is considered the last flood protection for building interiors. As such, a failure of an envelope component is likely to influence damage to other components, since water enters the building and causes damage to internal components. At the same time, further failures in the building envelope become less likely, since the water head differential between interior and exterior of the building reduces (as thoroughly explained later on). Figure 6.3 lists examples of structural and envelope components as well as non-structural and non-envelope components. Note that a component can be both structural and part of the building envelope.

	<i>Structural Components</i>	<i>Non-structural Components</i>
<i>Envelope Components</i>	Walls Roof structure Foundation ...	Windows Doors ...
<i>Internal Components</i>	Floors Internal walls ...	Heating Appliances Installations Contents ...

Figure 6.3 – Examples of component classification

Given their roles within the building, envelope and structural components are treated with special consideration in the present modelling approach. Mechanical failure from hydrostatic and hydrodynamic pressure is only considered to affect envelope components, since they can be subject to a water head differential and are directly exposed to water flow velocity. On the other hand, in analysing the collapse of a building following damage to an individual component, only structural components are

relevant. The set of all envelope components is  $C_{Env}$ ; the set of all structural components is  $C_{Stru}$ .

### 6.2.2 Hazard

Hazard event  $HZ$  is assumed to be characterised through following basic hazard variables: water depth  $H$ , water flow velocity  $V$  and flood rise rate  $R$ . Following Kelman (2002), the present modelling approach explicitly considers the water level inside the building,  $H_t$ , since it plays an important role in assessing mechanical failures in the building envelope.  $H_t$  is modelled as a function of the basic hazard variables  $H$ ,  $V$ ,  $R$  under consideration of the behaviour of the building envelope, i.e.  $H_t$  is a derived hazard variable.

During the flood rise process the building is subject to different load cases given by a combination of  $H$ ,  $H_t$  and  $V$ . Since it is not known at which point during the flood rise a critical load case occurs, the whole time series must be considered. The water depth  $H_t$ , where the subscript  $t$  indicates the time at which the variable is evaluated, is modelled as

$$H_t = \min(t \cdot R, H), \quad (6.2)$$

with  $0 \leq t \leq T$ , where  $t = 0$  is the starting time of the event and  $t = T$  is the time when water depth  $H$  and  $H_t$  have both reached their respective maximal value. The water recession process, as well as the flood event duration, is not explicitly considered.

### 6.2.3 Damage processes

In the present paper the two most relevant damage mechanisms are considered at a component level, namely damage from water contact and mechanical component failure due to hydrostatic and hydrodynamic pressure. Moreover, it is considered that, upon mechanical failure of a structural component(s), the whole system (i.e. the building) might collapse. Other damage mechanisms, such as foundation scour and buoyancy, are not considered, since they are only relevant in very particular situations (very high water flow velocity and a water-tight building envelope respectively).

The water head differential  $\Delta H = H - H_t$  is of great relevance for the assessment of mechanical failures in the building envelope, as it determines the hydrostatic pressure on envelope components. During the flood rise process, the envelope components are subject to a number of different load cases, which are a function of water depth  $H$ ,

flood rise rate  $R$  and water infiltration flow rate  $Q$  into the building. The latter is a function of the opening area  $A$  in the envelope, through which water can infiltrate the building. This, in turn, is a function of the damage state  $DM$  of envelope components. An illustration of a flood rise process with a hypothetical failure in the envelope is given in Figure 6.4. It exemplifies the relation between different variables over time, in particular how a failure in the envelope affects the water infiltration flow rate  $Q$  and therefore  $H_i$  and  $\Delta H$ .

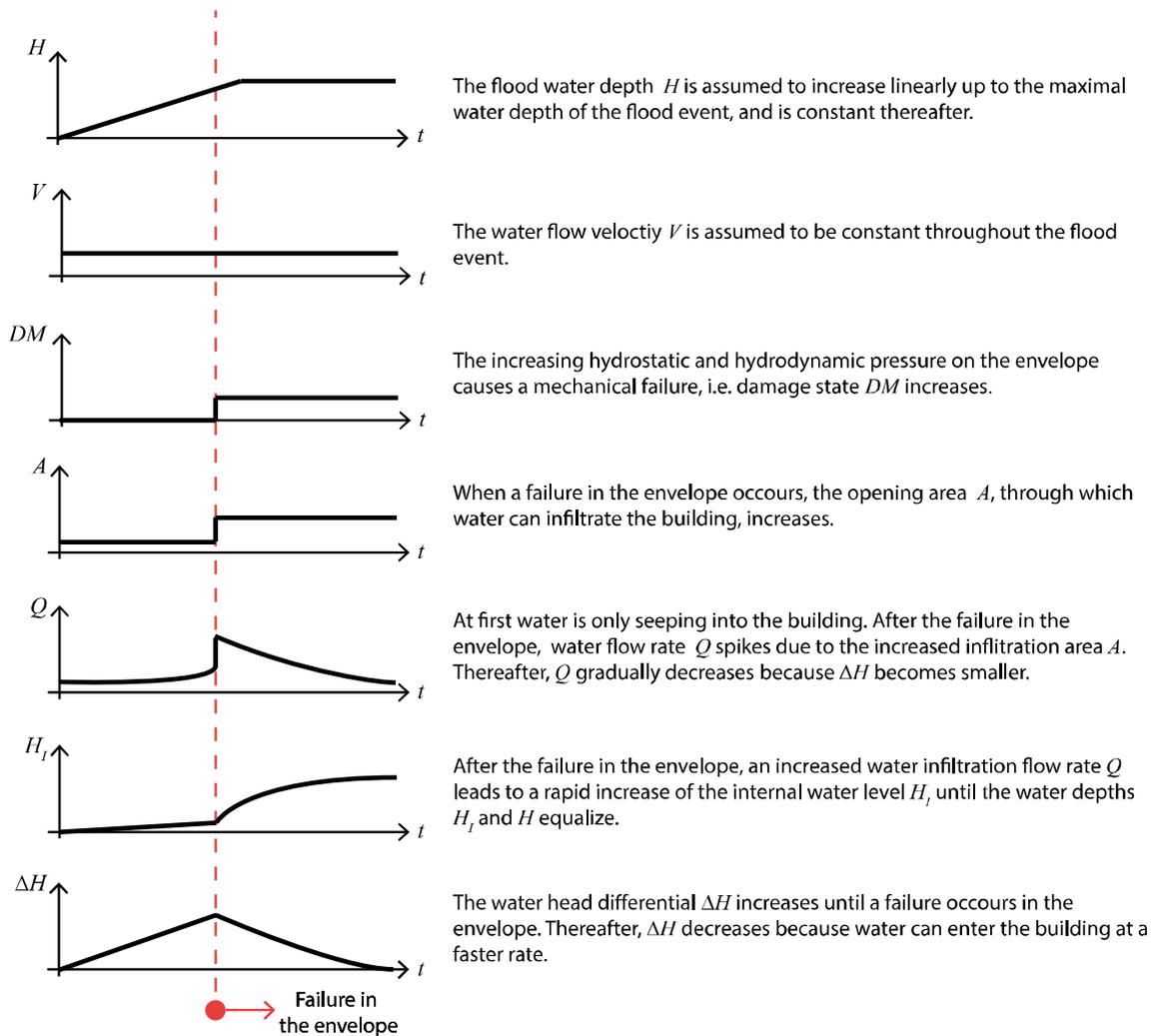


Figure 6.4 - Qualitative illustration of a flood rise process with failure in the envelope and the following increase in water infiltration and internal water depth

Following these observations, mechanical failures and water infiltration process are considered to be dependent and are modelled under consideration of the time process. Given these premises, the modelling tasks required to formulate a vulnerability model are summarised in Table 6.1.

Table 6.1 - Required model components to assess flood vulnerability of residential building

Model Component	Hierarchical level of consideration	State variable	Time process considered	Probabilistic characterisation
Mechanical failure model	Component	Damage state $DM_j$ , $j \in C_{Env}$ ; $\mathbf{DM} = (DM_j, j \in C_{Env})$ .	Yes	$P[\mathbf{DM}, H_I   HZ; \boldsymbol{\psi}, \mathbf{S}_\gamma]$
Water infiltration model	Component	Internal water depth $H_I$ .	Yes	
Water damage model	Component	Damage state $DW_j$ for all $C_j, j = 1, \dots, N$ ; $\mathbf{DW} = (DW_j, j = 1, \dots, N)$	No	$P[DW_j   H, H_I; \boldsymbol{\psi}, \mathbf{S}_\gamma]$
Structural system failure	System	System failure event $F$	No	$P[F   \mathbf{DM}; \boldsymbol{\psi}, \mathbf{S}_\gamma]$
Consequence model	System / Component	Monetary loss $L$	No	$P[L   \mathbf{DM}, \mathbf{DW}, F; \boldsymbol{\psi}, \mathbf{S}_\gamma]$

In the following, parameter vector  $\boldsymbol{\psi}$  and set  $\mathbf{S}_\gamma$  are omitted from conditional probability terms to allow a compact notation. Next to water depth  $H$ , variables  $H_I$ ,  $DM_j, j \in C_{Env}$  and  $\mathbf{DM}$  are modelled at different time steps in the flood rise process; the subscript  $t$  indicates the time at which the variable is evaluated. The subscript is omitted for  $t = T$ .

In the following sections the modelling of conditional probability terms in Table 6.1 is detailed.

#### 6.2.4 Mechanical failure and water infiltration

In this section the modelling of  $P[\mathbf{DM}, H_I | HZ]$  is detailed. The dependency between damage state vector  $\mathbf{DM}$  and internal water level  $H_I$  is modelled with a Markov Chain model.

The time process in Equation (6.2) is evaluated at discrete time intervals  $\Delta t$ , at each of which damage states  $DM_{j,t}, j = 1, \dots, N$  and internal water depth  $H_{I,t}$  are calculated with a water infiltration model and a mechanical failure model respectively. The probabilistic characterisation of the damage state  $DM_{j,t}$  is  $P_{DM_{j,t}} = P[DM_{j,t} | DM_{j,t-\Delta t}, H_{I,t-\Delta t}, H_{t-\Delta t}, V]$ ; the probabilistic characterisation of the internal water depth  $H_{I,t}$  is  $P_{H_{I,t}} = P[H_{I,t} | \mathbf{DM}_{t-\Delta t}, H_{I,t-\Delta t}, H_{t-\Delta t}, V]$ . Given  $H_t$  as per Equation (6.2) and postulating initial conditions  $H_{I,t} = 0$ ,  $V$  constant over the flood

rise process and that all components are in an undamaged state, the conditional probability term  $P[\mathbf{DM}, H_t | HZ]$  is modelled as:

$$P[\mathbf{DM}, H_t | HZ] = \sum_{\substack{\text{All values of} \\ \text{all marginalised} \\ \text{variables}}} \dots \sum \prod_{t=0}^T \left[ P_{H_{1,t}} \prod_{j=1}^N P_{DM_{j,t}} \right]. \quad (6.3)$$

Note that whereas  $P_{DM_{j,t}}$  and  $P_{H_{1,t}}$  are not explicitly conditional on  $R$ , they are implicitly, since  $H_t = f(t; H, R)$ . The assumption underlying Equation (6.3) is that the Markov condition holds, i.e. that  $DM_{j,t}$  and  $H_{1,t}$  are conditionally independent given variables at the previous time step, i.e.  $DM_{j,t-\Delta t}$ ,  $H_{1,t-\Delta t}$  and  $H_{1,t-\Delta t}$ . Also,  $DM_{j_1,t}$  and  $DM_{j_2,t}$  for  $j_1 \neq j_2$  are considered conditionally independent.

#### 6.2.4.1 Mechanical failure model

The mechanical failure model defines  $P[DM_{j,t} | DM_{j,t-\Delta t}, H_{1,t-\Delta t}, H_{1,t-\Delta t}, V]$ , i.e. the probability of mechanical failure at time  $t$  for given load case at time  $t - \Delta t$ .

In the general procedure, a finite set of  $K + 1$  mutually exclusive damage states,  $DM_j = dm_{jk}, k = 0, \dots, K$ , are considered for each component. A limit state function  $g_{jk}(\gamma_j, H_t, H_{1,t}, V)$  is formulated for each damage state. The probability of the component being in damage state  $k$  is evaluated as follows.

$$P[DM_{j,t} = dm_{jk} | H_{1,t-\Delta t}, H_{1,t-\Delta t}, V] = \begin{cases} 1 - P[g_{j1}(\cdot) < 0] & k = 0 \\ P[g_{jk}(\cdot) < 0] - P[g_{jk+1}(\cdot) < 0] & k = 1, \dots, K - 1 \\ P[g_{jk}(\cdot) < 0] & k = K \end{cases} \quad (6.4)$$

The evaluation of limit state function generally requires a structural analysis of component  $C_j$ . Concrete or masonry wall panels with out-of plane loads can be analysed with a variety of models available in literature, including FEM analysis (De Risi *et al.* 2013), yielding line analysis (Kelman and Spence 2003b) and 2D structural analysis (Roos 2003). Modelling approaches for the structural analysis of windows are found in Kelman (2002). Glass doors can be treated with the same model as windows; basis for modelling of non-glass doors is found in Kelman and Spence (2003a).

Note that  $DM_{j,t}$  is considered conditional on  $DM_{j,t-\Delta t}$  to assure that damage is non-decreasing with increasing time  $t$ . Nevertheless,  $DM_{j,t-\Delta t}$  is not considered in the structural analysis.

#### 6.2.4.2 Water infiltration model

The water infiltration model defines  $P[H_{I,t} | \mathbf{DM}_{t-\Delta t}, H_{I,t-\Delta t}, H_{t-\Delta t}, V]$ . The internal water depth is calculated as:

$$H_{I,t} = H_{I,t-\Delta t} + \frac{Q_t}{A_{floor}} \cdot \Delta t, \quad (6.5)$$

where  $A_{floor}$  is the floor area of the building and  $Q_t = \sum_{j \in \mathbf{C}_{env}} Q_{j,t}$ . Water flow rate  $Q_{j,t}$  through envelope component  $C_j$  is modelled with conditional probability term  $P[Q_{j,t} | H_{t-\Delta t}, H_{I,t-\Delta t}, DM_{j,t-\Delta t}, V]$ . The probabilistic characterisation of  $Q_t$  is found through convolution of  $P[Q_{j,t} | H_{t-\Delta t}, H_{I,t-\Delta t}, DM_{j,t-\Delta t}, V], j \in \mathbf{C}_{env}$ .

Modelling approaches for  $P[Q_{j,t} | H_{t-\Delta t}, H_{I,t-\Delta t}, DM_{j,t-\Delta t}, V]$  depend on the size of the opening area in component  $C_j$ , through which water flows into the building. Opening areas range from narrow cracks (e.g. wall crack) to large breaches (e.g. broken window). For crack flow literature, see e.g. Clarke *et al.* (1997) and Etheridge *et al.* (1977); for larger breaches Bernoulli's flow equation, with or without friction losses, can be utilised. Kelman (2002) provides a starting point for modelling of water infiltration between the two extreme cases.

#### 6.2.5 Damage from water contact

The great majority of flood damage is caused by water contact with components. The general conditional probability term for water damage  $DW_j$  to component  $C_j$  is  $P[DW_j | H, H_I]$ . Note that for the calculation of water contact only the water depth at time  $t = T$  is relevant. Despite the notation,  $DW_j$  is only conditional on one water depth,  $H$  or  $H_I$ , depending on whether component  $C_j$  is an envelope component or an internal component.

No general modelling approach to represent actual damage processes seems available in literature. However, a model can be established with the parametric vulnerability curves in Büchele *et al.* (2006), and the expert knowledge in Proverbs and Soetanto (2004).

### 6.2.6 Structural system failure

Mechanical failures are modelled at a component level, i.e. as local damages. Nevertheless, the possibility of progressive damage in the building as a consequence of local damage has to be considered. This type of analysis is typically found in robustness assessment; for an overview see Narasimhan (2012).

System failure event  $F$  is modelled conditionally on component damage states, i.e.  $P[F | \mathbf{DM}]$ . The assessment of  $P[F | \mathbf{DM}]$  is challenging: it may vary for different building configurations, entail several failure modes, and generally requires detailed structural analysis of the building for different realisations of  $\mathbf{DM}$ . An approximated approach is presented in the following. It assumes that each system failure mode is associated with damage to only one component. First, event  $F | DM_j$  is defined as system failure following damage to component  $C_j$ , whose occurrence probability is  $P[F | DM_j]$ . Second, the probability  $P[F | \mathbf{DM}]$  is assumed to be approximated with the assumption that all other components are undamaged. The probability of system failure is then approximated as:

$$P[F | \mathbf{DM}] = 1 - \prod_{j \in C_{\text{stru}}} (1 - P[F | DM_j]). \quad (6.6)$$

Equation (6.6) is exact when only one structural component is damaged or when two or more components are damaged, however no two damaged components contribute to the same system failure mode. When damaged components contribute to the same system failure mode, Equation (6.6) generally underestimates the probability of failure  $P[F | \mathbf{DM}]$ .

The probability  $P[F | DM_j]$  can be assessed through different approaches, e.g. expert judgement or engineering analysis. For the latter case, a possible approach is to perform a structural analysis of the whole building with component  $C_j$  in damage state  $DM_j = dm_{jk}$  and all other components undamaged, and calculate the probability of system failure  $F | DM_j = dm_{jk}$ , e.g. with reliability methods.

### 6.2.7 Consequence model

In the last modelling step, monetary losses  $L$  are calculated based on the damage states  $DM_j$  and  $DW_j$ , as well as system failure event  $F$ . Depending on whether a system

failure occurred, monetary loss  $L$  is determined with different approaches. In case of system failure, the full building value  $w_b$  is considered to be lost, i.e.  $P[L = w_b | F] = 1$ .

In the case no system failure occurred monetary loss  $L$  is the sum of component losses  $L_j$  for all components:

$$L = \sum_{j=1}^N L_j . \quad (6.7)$$

Monetary loss  $L_j$  to component  $C_j$  has probabilistic characterisation  $P[L_j | DM_j, DW_j]$  and can be modelled from empirical data (e.g. from insurance claims), through heuristics or expert knowledge, i.e. by considering costs of component repair strategies. The probabilistic characterisation of component loss  $L_j$  given hazard event  $HZ$  and that no system failure occurs is:

$$P[L_j | \bar{F}, HZ] = \sum \dots \sum_{\substack{\text{All values of} \\ \text{all marginalised} \\ \text{variables}}} P[L_j | DM_j, DW_j] P[DW_j | H_I] P[\mathbf{DM}, H_I | \bar{F}, HZ] . \quad (6.8)$$

where  $P[\mathbf{DM}, H_I | \bar{F}, HZ]$  is obtained from  $P[\mathbf{DM}, H_I | HZ]$  and  $P[\bar{F} | \mathbf{DM}]$  through Bayes' theorem. The probabilistic characterisation of  $P[L | \bar{F}, HZ]$  can be found by convolution of  $P[L_j | \bar{F}, HZ], j = 1, \dots, N$ , and its cumulative distribution function is

$$P[L \leq l | \bar{F}, HZ] = \int_0^{w_b} \left( I[l \leq x] P[L = l | \bar{F}, HZ] \right) dx , \quad (6.9)$$

where  $x$  is an integration variable and  $I[\cdot]$  is an indicator function which returns unity when the condition in the bracket is satisfied and zero otherwise. The cumulative distribution function of  $L$  in case of system failure  $F$  is:

$$P[L \leq l | F, HZ] = \begin{cases} 0, & l < w_b \\ 1, & l = w_b \end{cases} . \quad (6.10)$$

The cumulative distribution function  $P[L \leq l | HZ]$  of loss  $L$  given event  $HZ$  is obtained through weighted summation of  $P[L \leq l | \bar{F}, HZ]$  and  $P[L \leq l | F, HZ]$ , see Equation (6.11) and Figure 6.5.

$$P[L \leq l | HZ] = P[\bar{F} | HZ] \cdot P[L \leq l | \bar{F}, HZ] + P[F | HZ] \cdot P[L \leq l | F, HZ]. \quad (6.11)$$

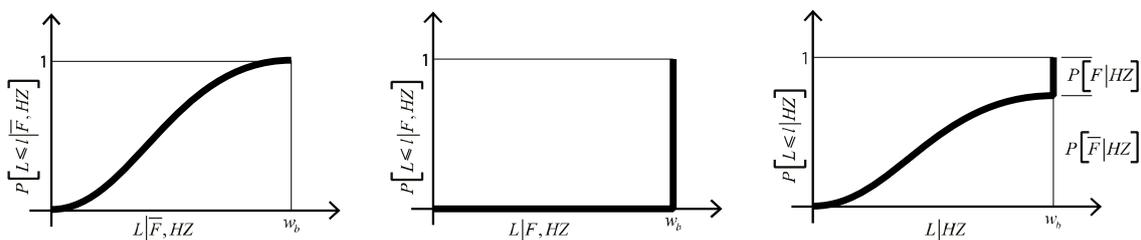


Figure 6.5 - Illustration of CDFs for  $L|\bar{F}, HZ$  on the left,  $L|F, HZ$  at the centre, and  $L|HZ$  on the right

An implementation of the approach, including parameterisation of a building and estimation of the conditional probability terms, is described in the next chapter.

### 6.3 Example implementation of the modelling approach

In the present chapter the modelling approach is implemented for a single family, 2-storey masonry building, as found throughout Europe, see Figure 6.6. The aim of the example implementation is to give detailed illustration of one possible application of the modelling approach.

In the example implementation all conditional probability terms are modelled and parametrised. However the logical linking of the conditional probability terms is not further detailed here, as it is fully defined in the modelling approach. Table 6.2 provides an overview of the main sub-models entailed by the modelling approach and how they are implemented in the present example implementation.

Table 6.2 - Overview of the modelling approach and its implementation

	<b>Modelling approach</b>	<b>Example implementation</b>	<b>Section</b>
Building parameterisation	The building is considered a collection of its components and is	- The list of considered components is given in Table 6.4.	6.3.1

	characterised with building parameter vector $\boldsymbol{\psi}$ and the collection of component parameter vectors $\mathbf{S}_\gamma = \{\gamma_1, \gamma_2, \dots, \gamma_N\}$ .	<ul style="list-style-type: none"> <li>- Considered building parameters are listed in Table 6.3.</li> <li>- Considered component parameters are listed in Figure 6.7.</li> </ul>	
Mechanical failure model	$P[DM_{j,t}   DM_{j,t-\Delta t}, H_{I,t-\Delta t}, H_{I-\Delta t}, V]$	<ul style="list-style-type: none"> <li>- The failure of walls is considered with a structural model according to Roos (2003).</li> <li>- The failure of windows is considered with a structural model according to Kelman (2002).</li> </ul>	6.3.2.1
Water infiltration model	$P[Q_{j,t}   H_{I,t-\Delta t}, H_{I-\Delta t}, DM_{j,t-\Delta t}, V]$	<ul style="list-style-type: none"> <li>- The opening area <math>A_j</math> in component <math>C_j</math> given <math>DM_j</math> is postulated.</li> <li>- The water flow rate through <math>A_j</math> is modelled with the Bernoulli equation for the flow of incompressible fluids.</li> </ul>	6.3.2.2
Water damage model	$P[DW_j   H, H_I]$	<ul style="list-style-type: none"> <li>- The expected value <math>E[DW_j] = f(H, H_I)</math> is modelled with a piecewise linear function.</li> <li>- Uncertainty of <math>DW_j</math> is modelled with a beta distribution.</li> </ul>	6.3.2.3
Structural system failure	$P[F   DM_j = dm_{jk}]$	<ul style="list-style-type: none"> <li>- The probability of failure <math>P[F   DM_j = dm_{jk}]</math> is postulated.</li> </ul>	6.3.2.4
Consequence model	$P[L_j   DM_j, DW_j]$	<ul style="list-style-type: none"> <li>- The value of components <math>W_j, j = 1, \dots, N</math> is modelled with a Dirichlet distribution.</li> <li>- The monetary loss for component <math>C_j</math> is determined as <math>L_j = f(DM_j, DW_j, W_j)</math>.</li> </ul>	6.3.2.5

In the following, the system representation scheme introduced in Section 6.2.1 is applied for the chosen building type, i.e. building parameters and component parameters are defined, and the list of considered components is detailed. Thereafter, in Section 6.3.2 models are specified to determine the conditional probability terms, which are part of the modelling approach. Note that hazard modelling is not detailed here as it is considered an input to the vulnerability model. The water depth  $H$  is measured from external terrain height  $h_{ref}$ , see Figure 6.6.

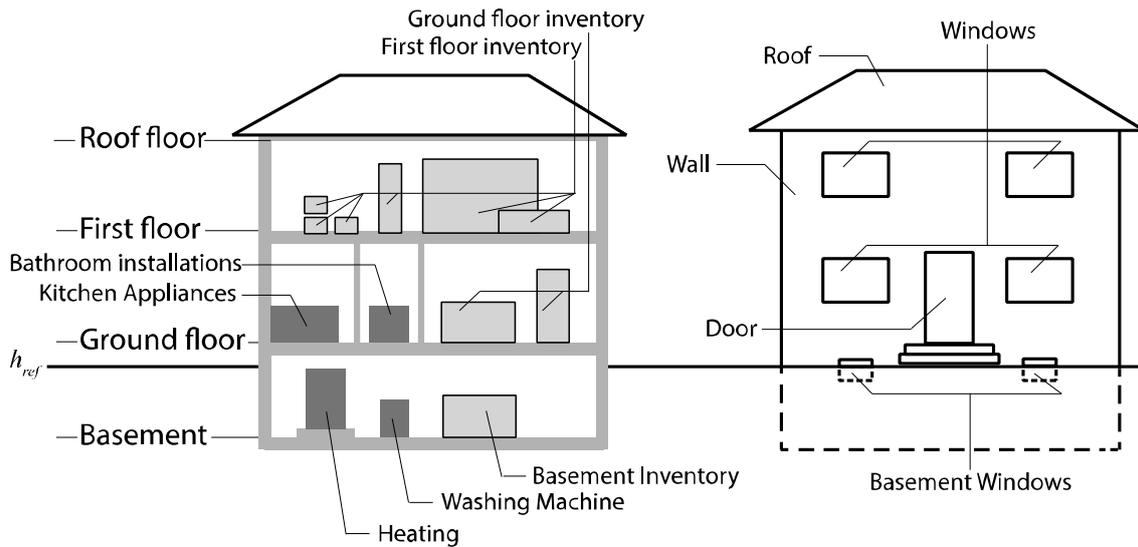


Figure 6.6 - Simplified representation of a 2-storey masonry building as found in continental Europe

### 6.3.1 Building parameterisation

In this implementation the building is parameterised with a high level of detail, i.e. many components are considered and each component is characterised by several parameters. The elements of the building parameter vector  $\psi$  are given in Table 6.3.

Table 6.3 - List of building parameters

	<b>Name</b>	<b>Description</b>
$n_s$	Number of stories	Number of stories excluding basement.
$w_b$	Building value	Monetary value of the building.
$a_{floor}$	Floor area	Floor area of one storey.
$h_{gf}$	Ground floor level	The height of the ground floor level above $h_{ref}$
$u$	Basement indicator	A Boolean parameter indicating the presence or absence of a basement.

The high level of detail entails a significant burden in defining the building and estimating parameters. In order to facilitate component parameterisation, three component classes  $\Xi_m, m=1,2,3$  are defined:  $\Xi_1$  for all internal components,  $\Xi_2$  for non-structural envelope components and  $\Xi_3$  for structural envelope components (refer to Figure 6.3 for examples). A class  $\Xi_m$  defines the elements of parameter vector  $\gamma_j$  by which each component  $C_j$ , instance of  $\Xi_m$ , is characterised. The parameters defined for each component class are shown in Figure 6.7. The list of considered components, as well as their classification, is given in Table 6.4.

$\omega$	The ratio of the component value to the building value.	Class [I] <sub>1</sub>	Class [I] <sub>2</sub>	Class [I] <sub>3</sub>
$h_{\min}$	Elevation over $h_{ref}$ of the component's bottom edge.			
$h_{\max}$	Elevation over $h_{ref}$ of the component's top edge.			
$\eta$	Water damage fragility threshold parameter.			
$\alpha$	Water damage fragility gradient parameter.			
$w$	Width of the component.			
$b$	Thickness of the component.			
$f_c$	Characteristic value of the compressive strength of the component material.			
$f_t$	Characteristic value of the tensile strength of the component material.			
$\mathbf{a}$	Parameter vector $\mathbf{a} = (a^{(0)}, a^{(1)}, a^{(2)}, a^{(3)})$ determines the components water infiltration area for given damage state, i.e. $A_j \left( DM_j = dm_{jk} \right) = a^{(k)}$ .			
$\mathbf{p}_f$	Parameter vector $\mathbf{p}_f = (p_f^{(0)}, p_f^{(1)}, p_f^{(2)}, p_f^{(3)})$ determines the probability of building failure given the component's damage state, i.e. $P \left[ F \mid DM_j = dm_{jk} \right] = p_f^{(k)}$ .			

Figure 6.7 - Component parameters for each component class

The component list in Table 6.4 is given in Penning-Rowsell *et al.* (2010) for a UK building, however, it is believed to be generally valid for 1-to-2 storey masonry building in Europe. Similar component lists can be found in BMVBW (2005). An alternative list considering only few components can be found in Nadal *et al.* (2010).

A modelled building can include several instances of each component in Table 6.4, e.g. a window component can be instantiated several times on the ground floor and on the upper floors to have a faithful representation of a building.

Table 6.4 - List of considered components with classification

Component Name	Component class	Component Name	Component class
Wall	$\Xi_3$	Heating radiators	$\Xi_1$
Roof structure		Electrical system	
Wall finishing	$\Xi_2$	Technical installations	
External doors		Hi-Fi/TV/Electrical goods	
Windows		Upholstered	
Basement Windows		Chipboard furniture	
Internal walls	$\Xi_1$	Polished wood furniture	
Internal wall finishing		Soft furnishing	
Internal doors and frames		Personal belongings	
Fitted furniture		Sanitary fittings	
Floor structure		Boiler	
Floor finishing		Heating firing unit	
Plumbing pipes			

### 6.3.1.1 Parameter assessment

A rationale for the assessment of parameter values and, if necessary, uncertainty is provided.

In engineering applications, many parameters are often not known with precision due to lack of knowledge (epistemic uncertainty) and to natural variability (aleatory uncertainty). In either case it is advisable to model a parameter as a random variable, i.e. considering parameter uncertainty. Depending on the type of analysis at hand, that is, whether a single building with known configuration or a portfolio of buildings, with incomplete knowledge on building configuration is analysed, the parameters that are considered as random variables may change.

There are generally three means of assessment: expert/engineering knowledge (including literature), statistical knowledge and direct measurement. Expert and engineering knowledge can be applied in the assessment of material properties, probabilities of failure, and fragility parameters for water damage. The assessment of geometry and monetary value parameters has to be differentiated by type of analysis. For the analysis of a specific (known) building, these can often be directly observed, e.g. measured or estimated with very good precision. For the analysis of a building, whose configuration is uncertain (e.g. when analysing a portfolio of buildings) the direct measurement becomes prohibitive and a statistical assessment is recommended.

As such, in a single building analysis, the geometrical features ( $a_{floor}$  of the building;  $h_{min}$ ,  $h_{max}$ ,  $w$ ,  $b$  of each component) are considered deterministic and the monetary values of components is modelled with small variance. In a portfolio analysis geometrical features differ between buildings and are generally not known with precision for all the buildings. Therefore, they are considered as random variables. Moreover, also due to lack of knowledge, the monetary value of components is modelled with larger variance. For both types of analysis, material characteristics ( $f_t$  and  $f_c$ ), water fragility parameters ( $\eta$  and  $\alpha$ ) as well as well opening area vector  $\mathbf{a}$  are modelled as random variables. In the present implementation of the modelling approach the building value  $w_b$ , component value weight  $\omega$  and the probability of failure vector  $\mathbf{p}_f$  are considered deterministically. The impact of parameter uncertainty on the results is significant, as will be shown in Figure 6.12.

Parameter values utilised for this implementation of the modelling approach are given in Table 6.10 and Table 6.11 in the Annex to Chapter 6. The uncertainty parameterisation for single building analyses and portfolio analyses is given in Table 6.12 in the Annex to Chapter 6.

### 6.3.2 Modelling of conditional probability terms

The present section describes the calculation of the conditional probability terms to model flood vulnerability of the building described in the previous section.

#### 6.3.2.1 Mechanical failure model

The modelling of  $P[DM_{j,t} | DM_{j,t-\Delta t}, H_{t,t-\Delta t}, H_{t-\Delta t}, V]$  is detailed. In principle, mechanical failure is considered for each envelope component, i.e. component classes  $\Xi_2$  and  $\Xi_3$ , including windows, walls, doors and roof. However, according to Kelman and Spence (2003a) the probability of mechanical failure of a door during a flood is negligible, since masonry wall panels generally fail before. Therefore, no mechanical failure model for doors is specified. Moreover, the mechanical failure of roof is not considered; it is assumed that if a flood would endanger the integrity of the roof structure, the building would fail before the water depth reaches the roof level. Therefore, the following analysis is only carried out for windows and wall components.

For each analysed component  $C_j$ , a set of  $K_j$  discrete damage states  $DM_j = dm_{jk}, k = 1, \dots, K_j$  and corresponding limit state functions  $g_{jk}(\cdot)$  are specified.

For wall components the postulated damage states and limit state functions are given in Table 6.5. A structural analysis of wall component is carried out according to Roos (2003) with a 2D structural analysis. The maximal loading moment  $M_s$  is calculated as illustrated in Figure 6.8. Note that water flow velocity  $V$  is considered constant over the water depth and, ground water is not considered. The limit state functions, given in Table 6.5, compare  $M_s$  to the wall cracking moment and ultimate moment, which are calculated as:

$$M_c = \frac{f_t + \sigma_N}{6} \cdot w \cdot b^2, \quad (6.12)$$

$$M_u = \frac{f_c - \sigma_N}{6} \cdot w \cdot b^2. \quad (6.13)$$

where  $\sigma_N$  is the compressive stress from the normal force in the wall. Only bending failures in the wall are considered. Wall openings, irregularities and horizontal load transfer in the wall are not considered in the analysis. Linear elastic material behaviour is assumed.

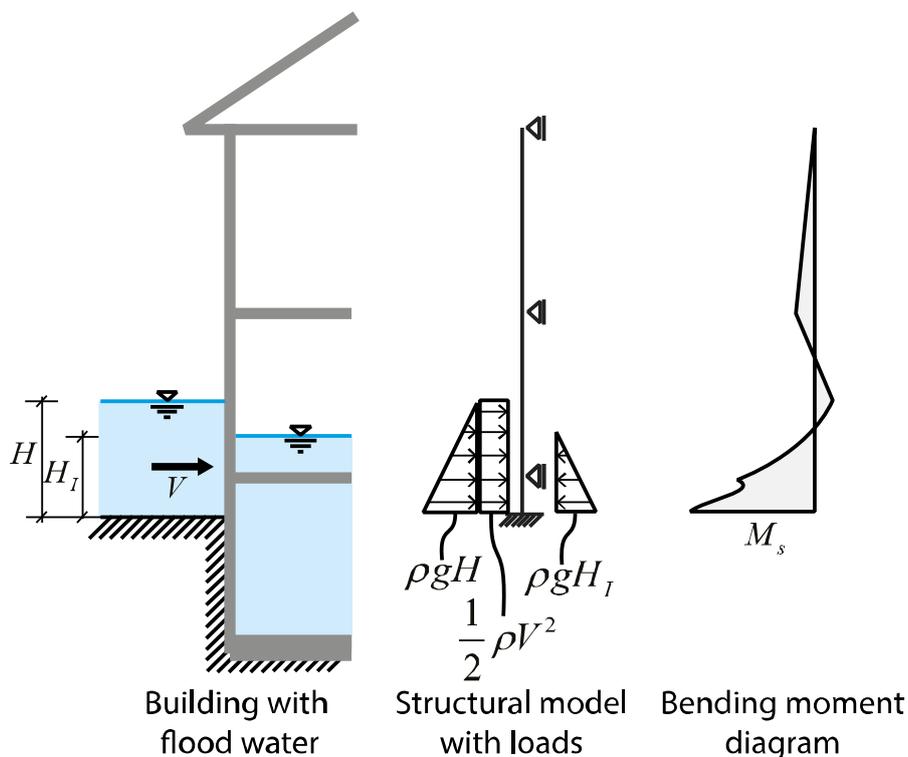


Figure 6.8 - Structural models utilised for the assessment of wall loading and bending moment diagram

Table 6.5 - Damage states and respective limit state function for wall components

Damage state $DM_j$		Limit state function
$dm_{j0}$	No damage	-
$dm_{j1}$	Cracking	$g_1 = M_c - M_s$
$dm_{j2}$	Partial collapse	$g_2 = M_u - M_s$
$dm_{j3}$	Collapse	$g_3 = 1.2M_u - M_s$ (The factor 1.2 is postulated and is justified by the fact that plasticity and horizontal load transfer were not considered in the structural analysis.)

Structural analysis of windows is modelled according to Kelman (2002). Bending failure of glass panels is considered under the assumption that glass panels are simply supported by the window frame. Horizontal and vertical load transfers in the window pane are considered. Only two damage states are modelled for windows as described in Table 6.6.

Table 6.6 - Damage states and respective limit state function for windows components

Damage state $DM_j$		Limit state function
$dm_{j0}$	No damage	-
$dm_{j1}$	Full damage	$g_1(\cdot) = f_t - \frac{6M_s}{wb^2}$

### 6.3.2.2 Water infiltration model

The water infiltration model  $P[H_{1,t} | \mathbf{DM}_{t-\Delta t}, H_{I,t-\Delta t}, H_{t-\Delta t}, V]$  is modelled through Equation (6.5) and  $P[Q_{j,t} | H_{t-\Delta t}, H_{I,t-\Delta t}, DM_{j,t-\Delta t}, V]$ . Here the modelling of  $P[Q_{j,t} | H_{t-\Delta t}, H_{I,t-\Delta t}, DM_{j,t-\Delta t}, V]$  is described.

As previously mentioned, crack flow should be treated separately from free water flow through large openings. Nevertheless, here only the Bernoulli equation for the flow of incompressible fluids is utilised to determine the water flow rate into the building. Friction and flow velocity  $V$  are neglected. The water flow rate  $Q_{j,t}$  through component  $C_j$  at time  $t$  is calculated as:

$$Q_{j,t} = A_j \cdot r_t \cdot \sqrt{2g\Delta H_t}, \quad (6.14)$$

where  $g$  is the gravitation constant,  $\Delta H_t = H_t - H_{I,t}$  and  $r_t$  is the proportion of the component under water, i.e.

$$r_t = \frac{\min(h_{\max}, H_t) - h_{\min}}{h_{\max} - h_{\min}}, \quad (6.15)$$

where  $h_{\min}$  and  $h_{\max}$  are the elevation of the components bottom and top edge above the external terrain level  $h_{ref}$  respectively. The opening area  $A_j$  in component  $C_j$  through which water can flow into the building is modelled conditionally on the damage state of the component, i.e.  $A_j | (DM_j = dm_{jk}) = a^{(k)}$ , where  $a^{(k)}$  is the  $k$ th element of parameter vector  $\mathbf{a}$ , see Figure 6.7.

### 6.3.2.3 Damage from water contact model

In this section the modelling of  $P[DW_j | H, H_t]$  is addressed. Due to the lack of data and models describing damage to components from water contact, a simple approach is postulated. Water damage state  $DW_j$  is considered a continuous variable that can take values between  $DW_j = 0$  (no damage) and  $DW_j = 1$  (full damage). The expected value  $E[DW_j]$  is modelled as follows:

$$E[DW_j | H, H_t] = \min\left(1, \max\left(\tan \alpha (H^* - \eta), 0\right)\right), \quad (6.16)$$

where  $H^* = H - h_{\min}$  for envelope components and  $H^* = H_t - h_{\min}$  for internal components. As illustrated in Figure 6.9, parameter  $\eta$  is a threshold value; when  $H^* > \eta$  the component is damaged. Parameter  $\alpha$  is the gradient of the linear fragility curve. The beta distribution is chosen to model uncertainty in  $DW_j$ , since the beta distribution is bounded between 0 and 1 (just as  $DW_j$ ), and since it is flexible to parameterise. Given  $E[DW_j]$ , the value of the variance  $Var[DW_j]$  is postulated as  $Var[DW_j] = 0.2 \cdot \left(0.5^2 - (0.5 - E[DW_j])^2\right)$ , which corresponds to 20% of the maximum variance allowed by the beta distribution. Given  $E[DW_j]$  and  $Var[DW_j]$  the beta distribution parameters can be estimated straightforwardly.

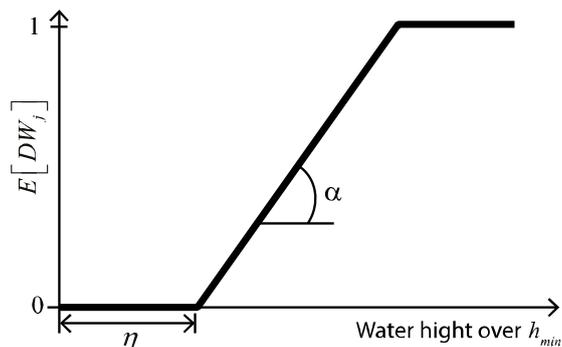


Figure 6.9 - Generic component fragility curve for water damage

#### 6.3.2.4 Structural system failure model

The probability  $P[F | DM_j = dm_{jk}]$  of building failure event  $F$  given that component  $C_j$  is in damage state  $DM_j = dm_{jk}$  is postulated with parameter  $p_f^{(k)}$  defined in Figure 6.7.

#### 6.3.2.5 Consequence model

The monetary loss in case of structural failure  $F$  is  $L = w_B$ . The probabilistic characterisation of component losses  $L_j, j = 1, \dots, N$ , i.e.  $P[L_j | DM_j, DW_j]$  is detailed in the following. The assumed general formulation of the monetary loss is:

$$L_j = \delta_j \cdot W_j \quad (6.17)$$

where  $\delta_j$  is the damage ratio of component  $C_j$ , i.e. the ratio between monetary loss  $L_j$  and component value  $W_j$ . The damage ratio  $\delta$  for the whole building follows from the definition of damage ratio and Equations (6.7) and (6.17).

The component damage ratio  $\delta_j$  is a function of  $DM_j$  and  $DW_j$ , and is postulated for envelope component as  $\delta_j = \min(\delta_{m,j} + DW_j, 1)$ , where  $\delta_{m,j}$  is damage ratio from mechanical failure (given in Table 6.7 for wall components and Table 6.8 for windows components), and for all other component as  $\delta_j = DW_j$ .

Table 6.7 - Postulated damage ratios  $\delta_{m,j}$  for given  $DM_j$  for wall components

Damage state $DM_j$	Damage ratio $\delta_{m,j}$
$dm_{j0}$	0
$dm_{j1}$	0.33
$dm_{j2}$	0.67
$dm_{j3}$	1

Table 6.8 - Postulated damage ratios  $\delta_{m,j}$  for given  $DM_j$  for window components

Damage state $DM_j$	Damage ratio $\delta_{m,j}$
$dm_{j0}$	0
$dm_{j1}$	1

Whereas the monetary value of the building  $w_B$  is assumed to be known, the value  $W_j$  of individual component  $C_j$  is uncertain. The variables  $W_j, j=1,2,\dots,N$  are “parts of a whole” as they are constrained by  $\sum_{j=1}^N W_j = w_B$ , i.e. they are of compositional nature and

are defined on a simplex. A probability distribution, which is well indicated to model compositional variables, is the Dirichlet distribution (Aitchinson 1986). Although its flexibility in modelling variance is limited and the covariance structure is strictly negative, the Dirichlet offers a compact formulation and simple parametrisation, and is thus utilised for this application. The vector  $\mathbf{W} = (W_1, W_2, \dots, W_N)$  is defined on the simplex  $S^N = \left\{ \mathbf{w} = (w_1, w_2, w_3, \dots, w_N) \in \mathbb{R}^N \mid w_j > 0, j = 1, 2, \dots, N; \sum_{j=1}^N w_j = w_b \right\}$  and is

modelled to follow the Dirichlet distribution with scaled support:

$$f_{\mathbf{W}}(\mathbf{w}; \boldsymbol{\omega}, \lambda, w_B) = w_B^{\left(1 - \lambda \sum_{j=1}^N \omega_j\right)} \frac{\Gamma\left(\lambda \sum_{j=1}^N \omega_j\right)}{\prod_{i=1}^N \Gamma(\lambda \omega_i)} \prod_{j=1}^{N-1} w_j^{\lambda \omega_j - 1} \left(w_b - \sum_{j=1}^{N-1} w_j\right)^{\lambda \omega_N - 1}, \quad (6.18)$$

with  $\Gamma(\cdot)$  the gamma function, value weight parameter  $\omega_j = E[W_j]/w_B$ ,  $\boldsymbol{\omega} = (\omega_1, \omega_2, \dots, \omega_N)$  and  $\lambda$  is a parameter controlling the variance in  $\mathbf{W}$ , see Custer and Nishijima (2012). It follows from this approach, that  $\sum_{j=1}^N W_j = w_B$  and  $E[W_j] = \omega_j \cdot w_b$ .

## 6.4 Results and comparisons

A Monte Carlo simulation is performed with the models detailed in Section 6.2 and 6.3. For each Monte Carlo sample, building and component parameters are sampled to obtain a sample building. For each sample building a vulnerability curve is obtained by calculating losses for hazard events  $HZ$  with different water depths  $H$ . If not otherwise stated,  $V = 0\text{ m/s}$  and  $R = 0.0001\text{ m/s}$  for each hazard event. Time interval is chosen to be  $\Delta t = 1000\text{ s}$ .

Most results shown in the following are for the example building described in Section 6.3.1, i.e., a 2-storey masonry building with basement. For comparison purposes, simulations are also made for buildings with the same component parameterisation, but with/without basement and 1- or 2-stories.

Furthermore, most results are for the assumption of incomplete knowledge of the building configuration (which is encountered in portfolio analysis where geometrical features are uncertain and the variance of component values is larger); when a building with known building configuration (geometrical characteristics are assumed to be known with precision and the component values are considered to have small variance) is considered, it is duly labelled.

In Section 6.4.1 vulnerability curves for different building parameterisation and hazard characteristics are presented. In Section 6.4.2, model results are compared to literature vulnerability models and damage data. Results are discussed in the next chapter.

### 6.4.1 Parameter studies

In this section model results for several parameterisations are presented, illustrating how building parameters and hazard characteristics influence vulnerability.

In Figure 6.10 the vulnerability curve for the 2-storey masonry building with basement is illustrated in terms of expected value of the damage ratio,  $E[\delta]$ , as well as the 10% and 90%-quantile values. In Figure 6.11 a comparison is made, between expected damage ratios of masonry buildings with different number of stories, and with or without basement. In Figure 6.12 the impact of uncertainty parameterisation on the vulnerability curve is illustrated. The comparison is made between vulnerability curves for a building with known building configuration and a building with incomplete knowledge of the building configuration. The two vulnerability curves are representative for a single building analysis and a single building in a portfolio analysis

respectively. Figure 6.13 illustrates the impact of different water flow velocities  $V$  on expected damage ratios.

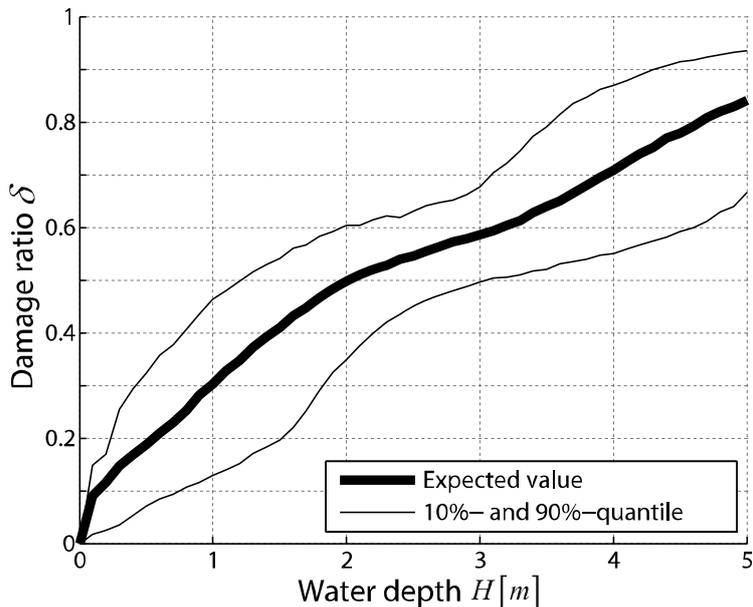


Figure 6.10 - Vulnerability curve with uncertainty bounds for a 2-storey masonry building with basement

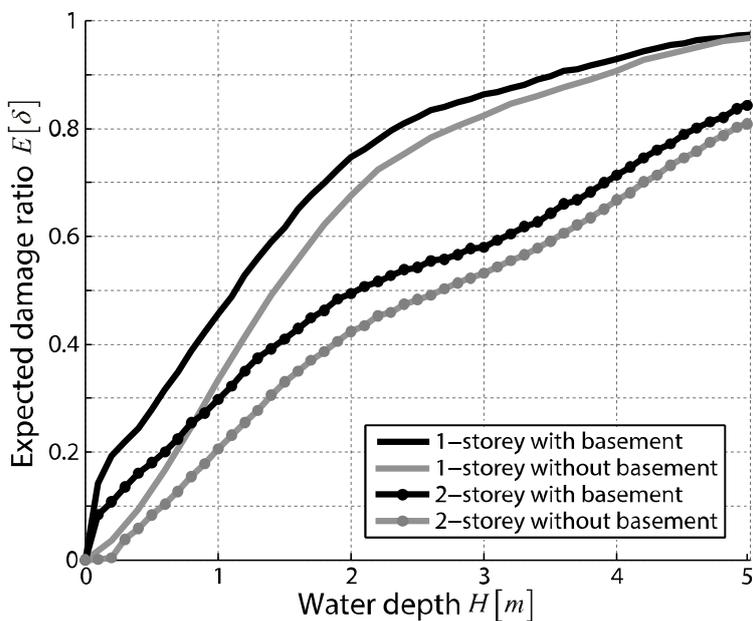


Figure 6.11 - Comparison of vulnerability curves for masonry buildings with different number of stories and with/without basement knowledge

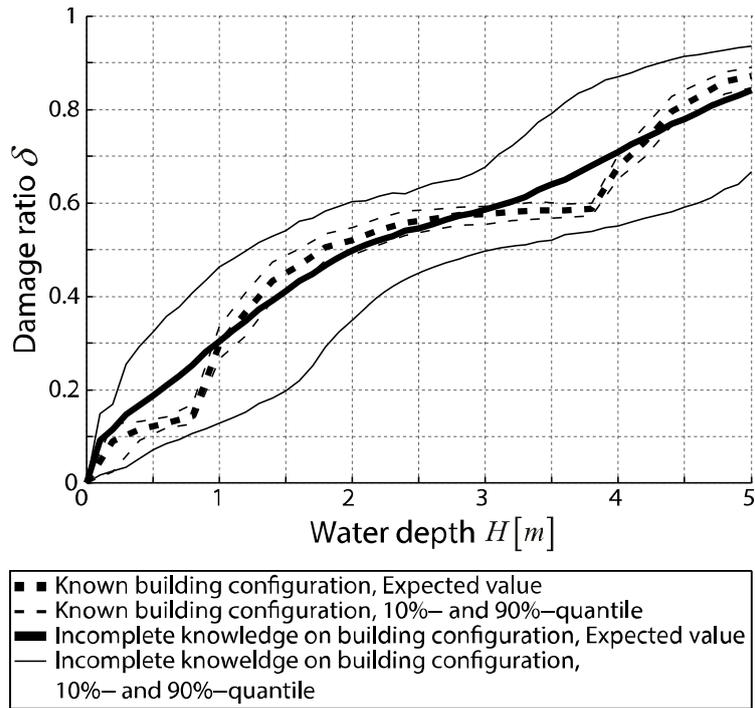


Figure 6.12 - Comparison of vulnerability curves with uncertainty bounds for a building with known configuration (small parameter uncertainty) and a building with incomplete knowledge on the building configuration (large parameter uncertainty)

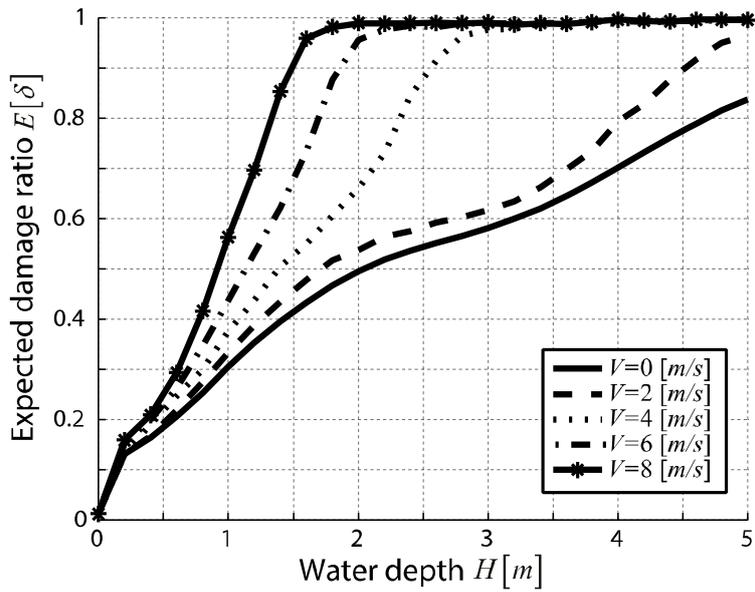


Figure 6.13.- Vulnerability curves for different water flow velocities  $V$

#### 6.4.2 Comparison with damage data and literature vulnerability models

A full validation of the presented model is not feasible due to its complexity and the lack of damage data in good quality. For partial validation, the model results are compared to vulnerability models available in literature and damage data from Germany.

In Figure 6.14 comparison is made with damage data from the German HOWAS database (HOWAS 2012). From the large database only relevant data points with good quality are considered, i.e. data points for 2-storey buildings with basement that included information on water depth and damage ratio.

In Figure 6.15 results are compared to vulnerability curve for “pre-1919 detached house” given in the appendix of the “Multi-Coloured Handbook”, MCH, (Penning-Rowsell *et al.* 2010), where vulnerability curves for short floods (less than 12 hours duration) and long floods (more than 12 hour duration) are specified.

A third comparison is made in Figure 6.16 with the vulnerability curves proposed in USACE (2003) for 1- and 2-storey buildings with basement and in USACE (2000) for 1- and 2-storey buildings without basement. For all USACE-curves it is assumed that the content value is 40% of the structure value. Note that these literature vulnerability curves are obtained through regression of damage data from past events. The construction material of the buildings underlying the vulnerability curves is not known and may include timber and masonry buildings.

Both MCH and USACE vulnerability curves are given as a function of water depth over ground floor level. The vulnerability curves of the present model are adapted for comparability. As a consequence, the water depth can be negative when water enters the basement, however does not reach the ground floor.

The final comparison is made in Figure 6.17, where the probability of building failure  $P[F]$  is plotted as contour lines in function of water depth  $H$  and water flow velocity  $V$ . The contour lines are compared to a damage criterion for “Total building destruction” for brick and masonry buildings by Clausen (1989).

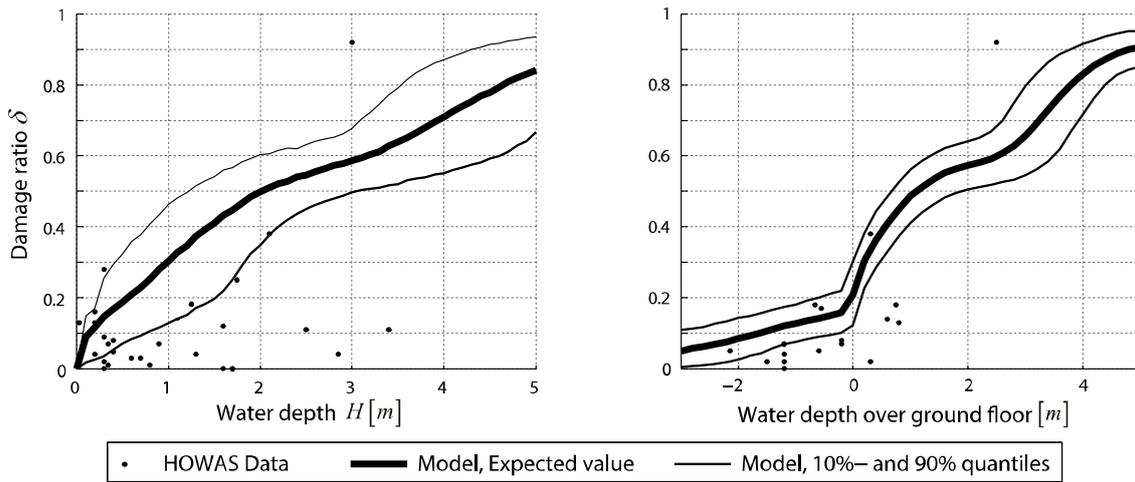


Figure 6.14 - Comparison of model results with damage data (HOWAS 2012) for two hazard indices: external water depth  $H$  (left) and water depth over ground floor (right)

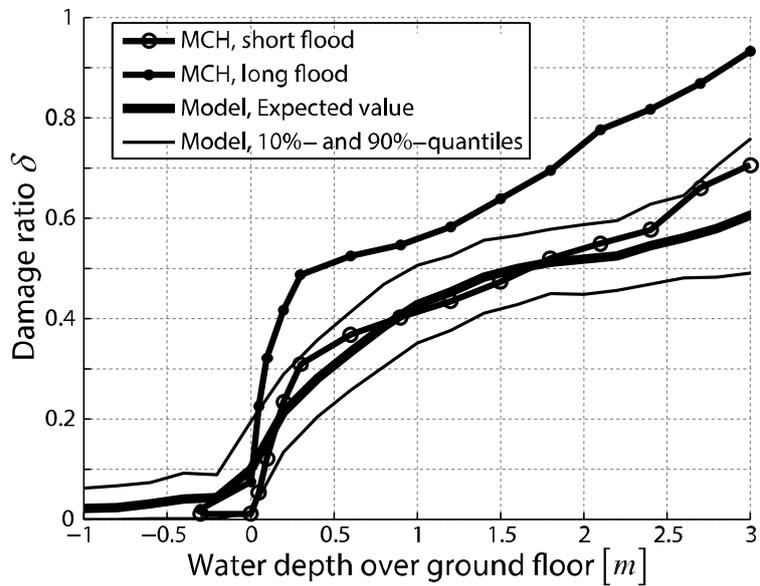


Figure 6.15 - Comparison of model results with MCH vulnerability curve from Penning-Rowsell *et al.* (2010)

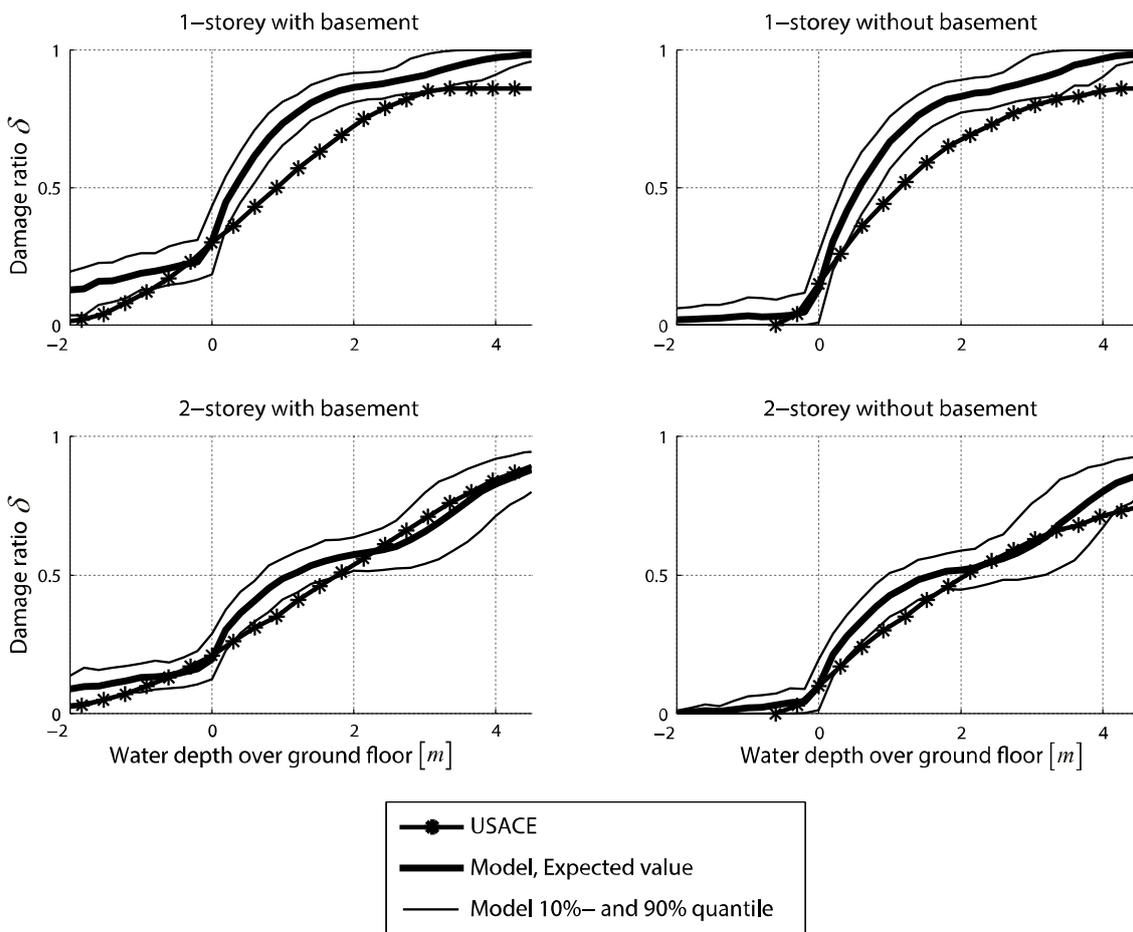


Figure 6.16 - Comparison of model result with vulnerability curves from USACE (2000) and USACE (2003)

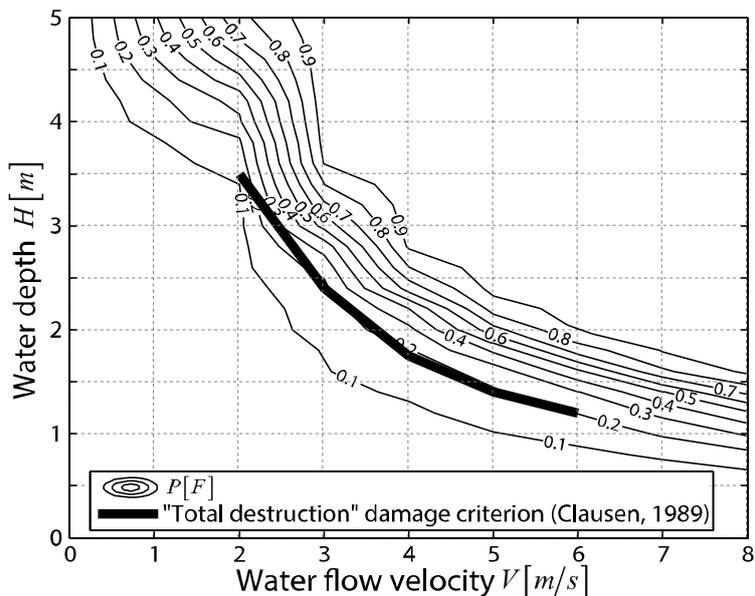


Figure 6.17 - Comparison of model result with Clausen (1989) damage criterion for “Total building destruction for brick and masonry buildings

## 6.5 Discussion

In the present chapter, the results are discussed and the advantages and shortcomings of the model are addressed in the context of the state of the art vulnerability models and the modelling objectives stated at the beginning of the paper.

### 6.5.1 Result discussion

The parameter studies generally indicate that the model can well capture the influence of hazard characteristics and building parameterisation on damage ratios.

In Figure 6.10 the expected damage ratio  $E[\delta]$  of a 2-storey building with basement steadily increases with water depth, however, the gradient of the curve is not constant. The changes in gradient are mainly due to a non-uniform distribution of components, and hence, monetary value, within each of the stories; i.e. components are concentrated in the lower part of each storey. For the same reason uncertainty bounds are not parallel to the expected damage ratio curve. It increases at water depths with large value concentration and decreases otherwise.

Figure 6.11 compares the vulnerability curves of different building types in terms of expected damage ratio  $E[\delta]$ . The presence of a basement significantly increases damage ratio compared to the same building without basement. The difference is particularly large for small water depth  $H$  and reduces with increasing water depth. 1-storey buildings consistently shows larger expected damage ratios compared to its 2-storey counterpart. The reason for this difference can be illustrated with additional study shown in Figure 6.18. Defining  $\delta_s$  as the damage ratio for a single storey analogously to the damage ratio  $\delta$  for the whole building, the expected damage ratio  $E[\delta_s]$  by storey is illustrated for a 1-storey building with basement (left) and a 2-storey building with basement (right). The basements and ground floors of the two buildings have very similar vulnerability curves. Therefore, the difference in vulnerability between 1-storey and 2-storey buildings observed in Figure 6.11 mainly arises, since 2-storey buildings have their value distributed over two stories rather than one. For instance, a total loss on the ground floor (approximately  $H = 3m$ ) results in a much higher damage ratio for the 1-storey building compared to the 2-storey building.

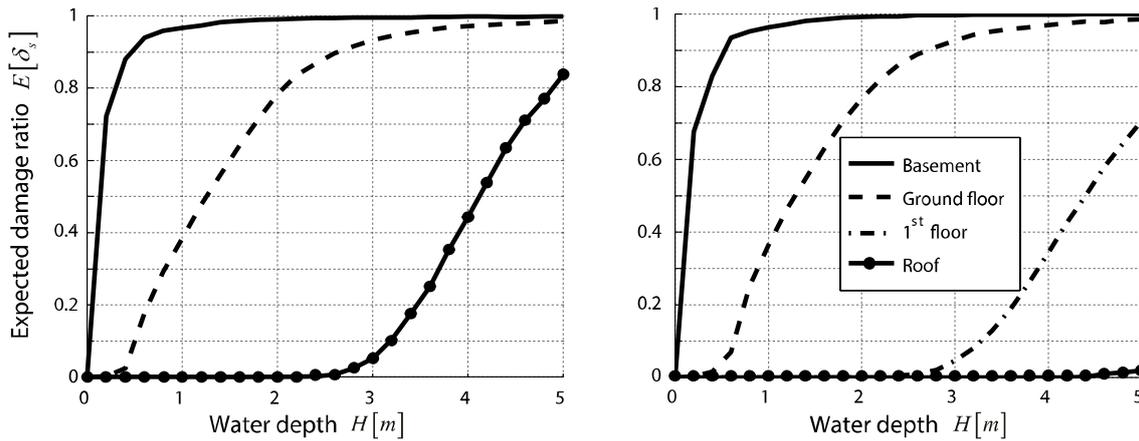


Figure 6.18 - Expected damage ratio  $E[\delta_s]$  for individual stories for 1-storey building with basement (left) and 2-storey building with basement (right)

According to Figure 6.12 parameter uncertainty has a major impact on the vulnerability curve and associated uncertainty bounds. Firstly, the uncertainty bounds are much larger when parameter uncertainty is large. Secondly, the changes in gradient of the expected damage ratio observed in Figure 6.10 are amplified when parameter uncertainty is reduced. The abrupt changes in gradient at  $H = 0.8\text{m}$  and  $H = 4\text{m}$  of the “Known building configuration” curves in Figure 6.12 correspond to ground floor level and first floor level of the modelled building respectively. Whenever the water reaches a higher storey, the damage ratio gradient increases as several components get watered, hence, damaged. In contrast, when the water rises to the upper parts of a storey, the damage gradient is small, since fewer components are situated in the upper part of a storey. When uncertainty is large this effect is less pronounced, since the levels of the floor, and therefore the value distribution, are considered uncertain.

Figure 6.13 illustrates the impact of water flow velocity. As expected, high water flow velocity  $V$  increases the damage ratios for given water depth  $H$ . Whereas the impact of water flow velocity is relatively minor at small water depth  $H$ , it becomes large as the water depth  $H$  increases. The reason for the increased expected damage ratio  $E[\delta]$  is that the probability of building failure event  $F$  increases remarkably with increasing water flow velocity  $V$ , see Figure 6.19.

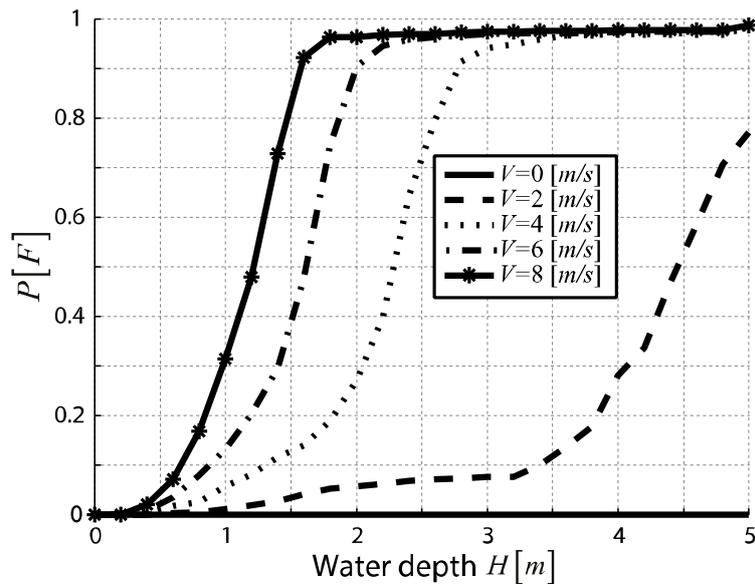


Figure 6.19 - Probability of building failure of a 2-storey masonry building with basement for different water flow velocities  $V$

In the following, the comparison of model results with empirical damage data and literature vulnerability models is discussed.

Figure 6.14 indicates that the model tends to overestimate damages. The variability of damage ratio for given water depth seems to be larger in the HOWAS (2012) dataset compared to the model result, and most damage data points lay outside the uncertainty bounds of the model result. Given these results, the HOWAS damage data cannot validate the model. However, it should be noted that the HOWAS data points are concentrated at small water depths, and therefore they do not allow evaluating the vulnerability curve at larger water depths.

As illustrated in Figure 6.15, the model results compare favourably with the vulnerability curve for short floods from Penning-Rowse *et al.* (2010), since the curve is within the uncertainty bounds of the model result. The vulnerability curve for long floods, however, lays outside the uncertainty bounds.

Figure 6.16 indicates that although the shape of vulnerability curves generally compare well, the model tends to overestimate damage ratios compared to USACE (2000) and USACE (2003). Whereas the USACE vulnerability curves for two storey buildings are mostly within the uncertainty bounds of the model result, this is not the case for 1-storey buildings.

As illustrated in Figure 6.17 the damage criterion for “Total building destruction” by Clausen (1989) approximately corresponds to the contour line for  $P[F]=0.2$ . Although it could be argued that the criterion should follow a contour line corresponding to a

larger or smaller probability of failure, the fact that it follows the contour line so closely implies that the impact of water flow velocity on building failures is considered in a comparable manner.

Summarising, the model seems to be qualitatively sound in terms of parameters sensitivity and yields comparable results to literature models. Nevertheless, further validation efforts have to be undertaken, possibly with better damage data and other literature models.

### 6.5.2 *Model discussion*

The presented vulnerability modelling approach allows for consideration of a large variety of building and hazard characteristics through explicit consideration of relevant damage processes. The considered damage processes are modelled with sub-models in terms of conditional probability. The modelling approach provides a guideline to logically link the sub-models. However, the modelling of individual sub-models is not prescribed in the modelling approach and may vary between implementations.

By considering buildings through its components and representing the most relevant damage processes, the modelling approach is in principle applicable to many building types and hazard situations, for which the represented damage processes are relevant (conversely, situations in which other damage processes, such as foundation scour and debris impact, are relevant, cannot be well represented). In an implementation of the modelling approach, component list and the specification of sub-models can be chosen in consideration of building type and the level of detail of the implementation. The possibility to apply the same modelling approach to a large variety of circumstances is seen as strength of the presented work. It allows for consistently developing vulnerability models for different building types and levels of detail. This is a distinct advantage over other vulnerability models, whose transferability in time and space is often limited (Cammerer *et al.*, 2013). Considering the building as a collection of components allows for a flexible representation of the spatial value distribution within the building (i.e. distribution of the components) and to model damage mechanisms at a component level. This is useful for the evaluation of the benefits of flood risk reduction measures. Flood proofing measures can be introduced in the building as additional components (e.g. a flood barrier which keeps water away from the building up to a certain height) or through reduction of the components fragility (e.g. when wet-proofing a building). The effect of early-warning systems can also be represented, in that

building contents can be moved to higher floors and ground floor openings can be sealed reducing the water infiltration flow rate into the building.

The use of conditional probability terms renders the modelling approach modular. Individual sub-models (i.e. one conditional probability term) can be replaced and improved when a more sophisticated sub-model becomes available, without need to change other parts of the modelling approach. Also, additional damage mechanisms can be considered with further conditional probability terms, without need of modifying other sub-models. Parameter uncertainty is considered in such a way that it is possible to consistently model the vulnerability of well-defined buildings with little uncertainty as well as the vulnerability of buildings within a portfolio, which is subject to larger uncertainties.

As mentioned in Mazzorana *et al.* (2014), for a meaningful analytical vulnerability study it is of great importance to accurately capture flood hazard impacting a residential building. While the modelling of hazard is not addressed in this paper, when implementing the modelling approach, hazard should be appropriately modelled in consideration of the level of detail of the implementation.

The implemented model is more complex than many other vulnerability models available in literature. Whereas model complexity is not necessarily negative, (see Schröter and Kreibich 2014) it raises the toll on establishing, implementing and validating the model. Development of individual sub-models, e.g. mechanical failures, water infiltration and water damage models, for each of the considered components is challenging and often requires expert judgement. Several postulations are made in the assessment of the large number of parameters and considered damage processes are implemented with simplified models. Furthermore, several hazard characteristics (e.g. flood duration, debris and contamination contents) and several damage mechanisms (e.g. buoyancy and foundation scour) are not considered.

Full validation of the modelling approach and its implementation, beyond the anecdotal attempt made in this paper is challenging, since it would require an analysis of results for a multitude of possible parameter combinations.

Further research works are recommended to improve and extend the model in the following. The applicability of the modelling approach for other building types, e.g. concrete or timber buildings should be demonstrated. The individual damage mechanism and water infiltration models should be individually refined and validated. A rationale for the estimation of the large number of parameters should be established. A sensitivity analysis in regard to the number and the type of considered components

should be made. The damage mechanisms that are not considered should be included in the modelling approach. Furthermore, it is recommended to improve the modelling of the hazard event, and test the modelling approach with real hazard event time series as a hazard input.

Despite the shortcomings and identified challenges, the proposed modelling approach and its implementation are an important step towards the establishment of a vulnerability model that can serve as a basis for engineering decision making for flood proofing of residential buildings.

## **6.6 Summary and conclusion**

In order to quantify the benefits of applying flood proofing measures to a residential building, a flood risk analysis is necessary. Furthermore, to compare the benefits of different flood proofing measures, the vulnerability model needs to accommodate the effect of the different flood proofing measures, i.e. decision alternatives, on consequences.

Such vulnerability models seem not to be available in literature. Due to the complexity of flood damage processes most vulnerability models are based on regression, only consider few building and hazard characteristics as explanatory variables, and cannot reflect the vulnerability reduction introduced by flood proofing measures.

In the present paper a new vulnerability modelling approach is proposed. The building is considered a collection of components, and fragility is modelled at a component level for two damage mechanisms: mechanical failure due to hydrostatic and hydrodynamic pressure, and damage from water contact. Moreover, it is considered that the building may collapse upon mechanical failure of a structural component. To accurately model damages, the water infiltration process into the building is modelled in detail and fragility is represented with engineering models where possible.

The modelling approach is defined in terms of conditional probability terms in a general manner, which should be applicable to a large variety of building types. Furthermore, the implementation of the modelling approach for a 2-storey masonry building with basement is detailed.

In a parameter study the impact of different hazard and building parameters on vulnerability is analysed. Moreover, results are compared to literature vulnerability models and to damage data from Germany.

The model shows the expected sensitivity to changes in building and hazard parameterisation and results are comparable to literature vulnerability models.

As such, the general model framework is considered valid to model vulnerability of residential building in floods. Its flexibility and reliance of engineering models make it well suited to be utilised as a basis for engineering decision making for flood proofing of residential buildings. Shortcomings of the model are thoroughly discussed; they are generally related to the complexity of establishing, implementing and validating the model. The presented modelling approach provides a strong foundation for future development and can be improved over time.

**Annex to Chapter 6**

Table 6.9 - List of abbreviations utilised in Chapter 6

<b>Abbreviation</b>	<b>Definition</b>
$A_{floor}$	Building floor area.
$C_j, j = 1, \dots, N$	The $j$ th building component.
$C_{Env}$	Set of all envelope components.
$C_{Str}$	Set of all structural components.
$D$	Damage state.
$DM_j$ and $DM_{j,t}^*$	Damage state of component $C_j$ from mechanical damage. May take values $DM_j = dm_{jk}, k = 0, \dots, K$ .
$DM$ and $DM_t^*$	$DM = (DM_1, DM_2, \dots, DM_N)$ contains damage states from mechanical failure for all components.
$DW_j$	Damage state of component $C_j$ from water contact.
$DW$	$DW = (DW_1, DW_2, \dots, DW_N)$ contains damage states from water damage for all components.
$F$	System failure event.
$g_{jk}(\cdot)$	Limit state function for component $C_j$ and damage state $DM_j = dm_{jk}$ .
$H$ and $H_t^*$	Water height external to the building.
$H_I$ and $H_{I,t}^*$	Water height internal to the building.
$HZ$	Hazard event.
$L$	Monetary loss to the building.
$L_j, j = 1, \dots, N$	Monetary loss to component $C_j$ .
$N$	Number of building components.
$N_b$	Number of building parameters.
$N_j$	Number of component parameters for component $C_j$ .
$Q$ and $Q_t^*$	Water infiltration flow rate through the whole envelope.
$Q_j$ and $Q_{j,t}^*$	Water infiltration flow rate through component $C_j$ .
$R$	Water rise rate.
$S_\gamma$	Collection of all component parameter vectors $S_\gamma = \{\gamma_1, \gamma_2, \dots, \gamma_N\}$
$t$	Time $0 \leq t \leq T$ .

Table 6.9 (continuation) - List of abbreviations utilised in Chapter 6

Abbreviation	Definition
$T$	$t = T$ is the time when water depth $H$ and $H_I$ have both reached their respective maximal value.
$V$	Water flow velocity.
$w_b$	Building value.
$\gamma_i, i = 1, \dots, N_j$	Component parameters.
$\Upsilon_j$	$\Upsilon_j = (\gamma_1, \gamma_2, \dots, \gamma_{N_j})$ contains the component parameter for component $C_j$ .
$\Delta H$	Water head differential between building interior and exterior $\Delta H = H - H_I$ .
$\Delta t$	Time step.
$\psi_i, i = 1, \dots, N_b$	Building parameter.
$\Psi$	$\Psi = (\psi_1, \psi_2, \dots, \psi_{N_b})$ contains all building parameters.

\* The subscript  $t$  indicates the time at which the variable(s) is evaluated. The subscript is omitted for  $t = T$ .

Note that additional parameters and variables defined in the example application are summarised in Table 6.3 and Figure 6.7.

Table 6.10 - Building parameters

	$n_s$	$w_b$ [USD]	$a_{floor}$ [ $m^2$ ]	$h_{gf}$ [m]	$u$
2-storey masonry building	2	1000000	150	0.8	1

## Flood vulnerability assessment of residential buildings (Paper 2)

Table 6.11 – Parametrisation of components. For components present on several floors, the ground floor parametrisation is provided

	Class	$h_{\min}$ [m]	$h_{\max}$ [m]	$\eta$	$\alpha$	$b$ [m]	$w$ [m]	$f_c$ [MPa]	$f_t$ [MPa]	$a$ [m <sup>2</sup> ]	$P_f$
Wall	[E] <sub>3</sub>	0	4	0.5	30	0.23	1	14	0.24	(0,1,0.3,5,20)	(0,0,0.1,0.6)
Wall finishing	[E] <sub>2</sub>	0	4	0.2	45						
Roof	[E] <sub>3</sub>	4	5	0.2	75					(1,2,5,20)	(0,0,0,0.1)
External doors	[E] <sub>2</sub>	0	2.6	0.2	75	0.04	0.8			(0.01,1)	
Windows	[E] <sub>2</sub>	1	2.8	0.2	75	0.004	1		10	(0.01,1)	
Internal walls	[E] <sub>1</sub>	0	4	0.2	30						
Internal wall finishing	[E] <sub>1</sub>	0	4	0.1	45						
Internal doors and frames	[E] <sub>1</sub>	0	2.6	0.2	75						
Fitted furniture	[E] <sub>1</sub>	0	1.6	0.2	75						
Floor structure	[E] <sub>1</sub>	0	0.6	0.5	90						
Floor finishing	[E] <sub>1</sub>	0	0.62	0.01	90						
Plumbing pipes	[E] <sub>1</sub>	0	1	2	20						
Sanitary fittings	[E] <sub>1</sub>	0	1.6	1	90						
Heating radiators	[E] <sub>1</sub>	0.1	1.2	1	90						
Electrical system	[E] <sub>1</sub>	0	4	0.01	90						
Technical installations	[E] <sub>1</sub>	0	4	0.01	90						
HiFi/TV/ Electrical goods	[E] <sub>1</sub>	0.5	1.4	0.01	90						
Upholstered	[E] <sub>1</sub>	0	1.6	0.01	90						
Chipboard	[E] <sub>1</sub>	0	1.2	0.2	90						
Polished wood	[E] <sub>1</sub>	0	1.8	0.2	90						
Soft furnishing	[E] <sub>1</sub>	0	2	0.01	90						
Personal belongings	[E] <sub>1</sub>	0	2	0.01	45						
Boiler	[E] <sub>1</sub>	-2.2	-0.5	1	90						
Heating	[E] <sub>1</sub>	-2.2	-0.5	0.2	90						

Table 6.12 - Uncertainty parameterisation

Parameter	Expected value		Distribution family	COV	
	Single risk	Portfolio		Single risk	Portfolio
$\lambda$	10000	100	-	-	-
$h_{gf}$	- <sup>1</sup>		Gamma	0.001	0.8 <sup>2</sup>
$a_{floor}$	- <sup>1</sup>		Lognormal	0.001	1 <sup>2</sup>
$f_c$	- <sup>1</sup>		Lognormal	0.17	0.17 <sup>3</sup>
$f_t$	- <sup>1</sup>		Lognormal	0,30	0,30 <sup>3</sup>
$\eta, \alpha$	- <sup>1</sup>		Normal	1	1
$w, b, h_{min}$ $, h_{max}$	- <sup>1</sup>		Normal	0.01	0.5
<b>a</b>	- <sup>1</sup>		Truncated Normal	0.1	0.1

<sup>1</sup> – The expected value is given in the parameter tables above.

<sup>2</sup>- Source: Statistical analysis of HOWAS (2012).

<sup>3</sup> – Source: JCSS (2001).

All other values are postulated.



## **7 Modelling and optimisation of a hierarchical flood protection system**

The present chapter describes an example application of the methodologies introduced in Chapter 4 and Chapter 6. A hierarchical flood protection system with three hierarchy levels (dike, local flood barriers and dry-proofing of individual buildings) is proposed and optimised for a small village in Switzerland. The aim of the example is to further illustrate the concept of hierarchical flood protection systems and the proposed methodologies. However, due to insufficient data quality and several assumptions and postulations the results are not of immediate practical relevance.

## 7.1 Introduction

The example application is carried out for the village of Kriessern, located in the Canton St. Gallen in Eastern Switzerland (see Figure 7.1); the river Rhine flows in close proximity to the village. The Rhine has its source in the central Alps, and feeds into the North Sea in The Netherlands. The part of the Rhine relevant to this study is called Alpine Rhine, which leads from Reichenau to the Lake Constance and in particular the “international section” of the Alpine Rhine, which forms the border between Switzerland and Austria, is of interest.

Over the last centuries, this river section has seen several course corrections and improvements to the flood protection system. The current flood protection system was constructed in early 20th century (Schenk *et al.* 2014); its main protection structure is a dike designed to protect the flood plain from river discharges of up to  $3100m^3/s$ , corresponding to a 100 year event. The dike has been breached several times in the past, most recently in Fussach in 1987 (IRR 2012).

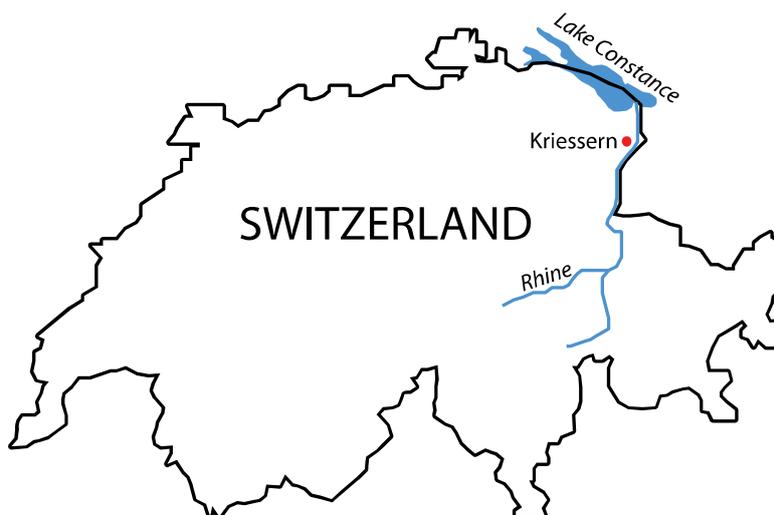


Figure 7.1 - The location of Kriessern in Northeastern Switzerland

The ageing dike, continued flood plain urbanisation and increased flood protection requirements have prompted governments of Austria and Switzerland to plan improvements to the flood protection system, with the goal of increasing protection for events with a river discharge of at least  $4300m^3/s$ , equivalent to a 300 year event, with a contingent plan for extreme events with discharges up to  $5800m^3/s$ . Several improvements to the flood protection system are currently being planned or implemented (IRR 2012, IRKA 2014, Schenk *et al.* 2014).

## 7.2 Study area

The international section of the Alpine Rhine is chosen for this example application due to the current state of its flood protection system. Kriessern is chosen because of its proximity to the Rhine and its small size, which is ideal for testing the implementation of the proposed methodologies. The study area is chosen to include a section of the river upstream and downstream of the village, see Figure 7.2.

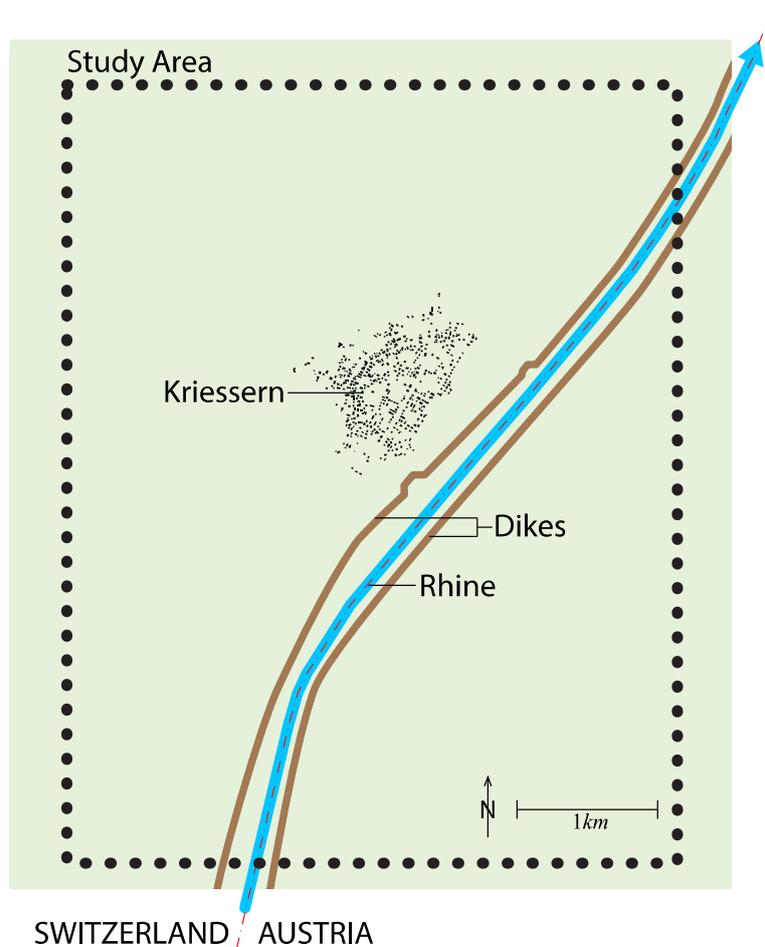


Figure 7.2 - Study area with river Rhine, dikes and Kriessern (Data: Swisstopo 2010)

At the end of 2013, Kriessern had 1738 inhabitants (Politische Gemeinde Oberriet 2014). Its economy includes small industrial, commercial and tourist sectors. Next to the village itself, the study area mainly includes agricultural land and small patches of forest.

### 7.2.1 *Scope of analysis*

The goal of this analysis is to identify an appropriate flood protection system for Kriessern. Taking the current dike as a starting point, a hierarchical flood protection system with three hierarchy levels is proposed and the optimised through a risk-based decision analysis.

While the Austrian side of the Rhine Valley is included in the flow routing model, consequences are modelled only on the Swiss river bank. Flood consequences to agriculture, forest, industry and infrastructure are disregarded.

### 7.2.2 *Data*

The location of buildings and infrastructure (including the dike) in the study area is available from Swisstopo (2010), which also provides information on the floor area of buildings. Several further building characteristics are available as aggregate statistics from BFS (2014). As the exact location of receptors is known, the probabilistic disaggregation model introduced in Chapter 5 is not necessary to model the spatial distribution of receptors.

The digital elevation model (DEM) for the study area is available from Swisstopo (2005) at a 25m resolution, with an average error in the elevation value of 1.5m. Profile and cross-section data for the river, as well as geometrical and resistance features of the dike, are not available.

Several water gauges are present on the Alpine Rhine within or in proximity of the study area, i.e. in Oberriet and Diepoldsau on the Swiss river bank (BAFU 2014a) and in Bangs on the Austrian river bank (Land Vorarlberg 2014). A recent discharge history is available for each station. For Diepoldsau, the annual maximum discharge data from 1919 to present is also available.

### 7.2.3 *Current flood protection system*

Currently Kriessern is protected by the earth dike built at the beginning of the 20th century. The resulting single-structure flood protection system is schematically illustrated in Figure 7.3. As mentioned, no details on the geometry and structural resistance of the current dike are available. The current dike height is assumed to be  $h_{dike}=3m$  above terrain; this assumption is based on the discharge-stage relationship in Diepoldsau available from BAFU (2014b) and the information that the dike is overtopped when river discharge reaches  $3100m^3/s$  (Schenk *et al.* 2014).

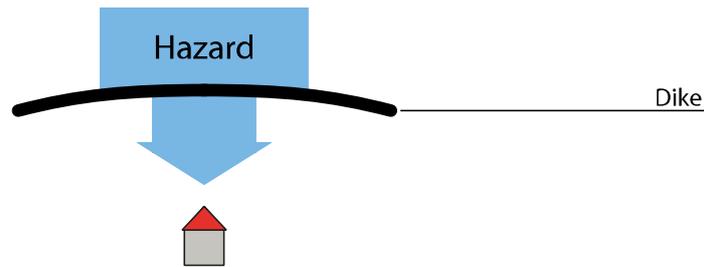


Figure 7.3 - Schematic representation of the current flood protection system in Kriessern

Different sources indicate that the dike is in poor conditions, as earth cavities have been repeatedly discovered (Rhesi 2012), and the dike has been breached on several occasions, most recently in 1987 in Fussach (Schenk *et al.* 2014). For this reason, the breaching probability of the current dike is assumed to be high (see fragility model in Section 7.3.2.1).

#### 7.2.4 Proposed hierarchical flood protection system

A hierarchical flood protection system with three hierarchy levels is proposed. The first hierarchy level is formed by the dike, the second level by local flood barriers and the third level by building envelopes, which protect the interiors of buildings; dry-proofing is considered as a measure to improve building envelopes. The proposed hierarchical flood protection system is schematically illustrated in Figure 7.4. In the next sections, the decision alternatives considered on each hierarchy level are detailed.

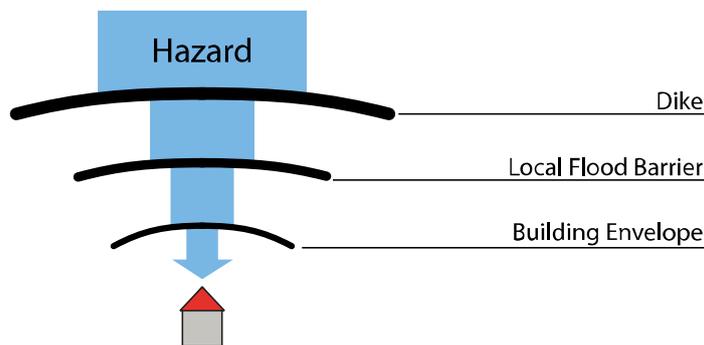


Figure 7.4 - Schematic representation of a hierarchical flood protection system considered for Kriessern

### 7.2.4.1 Dike

Next to the current dike configuration, two additional alternative configurations are considered. Alternative 1 entails a dike with the same height as the current dike, but with increased structural resistance. Alternative 2 entails a dike with both increased height and increased structural resistance. Figure 7.5 illustrates the three considered configurations.

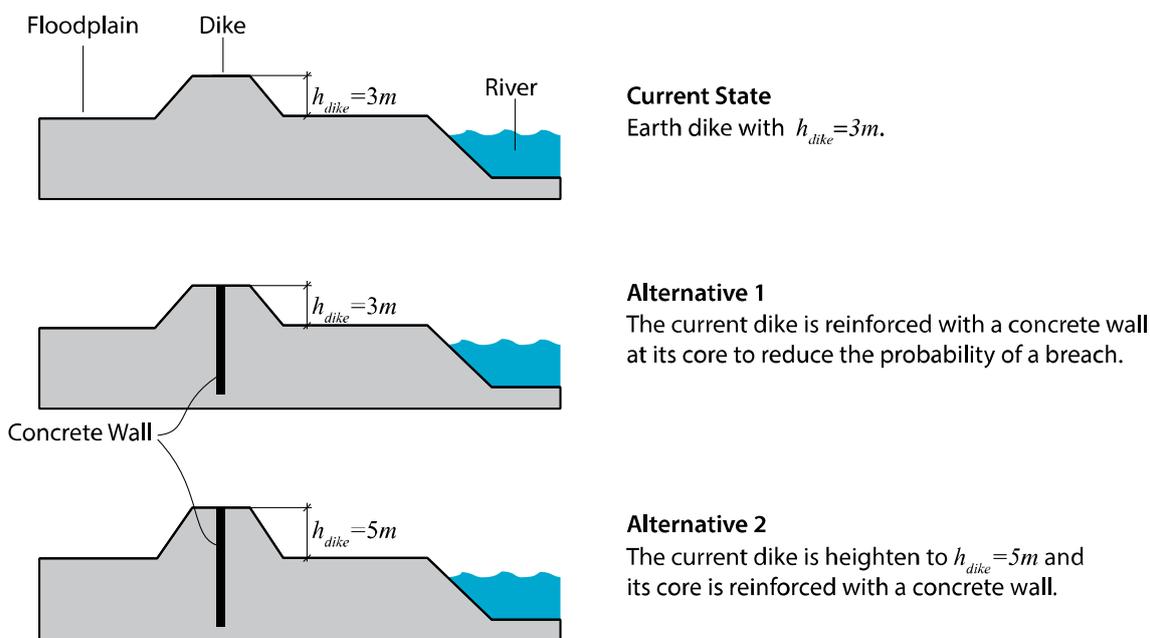


Figure 7.5 - Illustration of the considered dike decision alternatives

On the first hierarchy level, the set of decision alternatives available to the decision maker is  $A_1 = \{a_1^{(0)}, a_1^{(1)}, a_1^{(2)}\}$ . Decision alternative  $a_1^{(0)}$  is the current dike, and  $a_1^{(1)}$  and  $a_1^{(2)}$  are Alternative 1 and Alternative 2 respectively.

The dike section protecting Kriessern is assumed to be  $6km$  long. Construction costs entailed by each decision alternative are summarised in Table 7.1. Construction costs for  $a_1^{(1)}$  are derived from IRKA (2014); construction costs for  $a_1^{(2)}$  are derived from Delcan (2012). Life time of the dike is considered to be  $T_1 = 100$  years for all decision alternatives; construction time is disregarded. Inspection, maintenance and decommissioning costs are not considered in this analysis.

Table 7.1 - Construction costs for considered dike alternatives

Decision alternative	Construction cost per <i>km</i> [CHF] <sup>12</sup>	Total construction costs [CHF]
Current state - $a_1^{(0)}$	0	0
Alternative 1 - $a_1^{(1)}$	217'000	1'302'000
Alternative 2 - $a_1^{(2)}$	1'500'000	9'000'000

#### 7.2.4.2 Local flood barrier

As detailed in Section 3.5.2, local flood barriers are available in a plethora of forms, with different protection heights and resistance specifications; Ogunyoye *et al.* (2011) give an overview of commercially available products, providing structural and operational details, as well as installation costs of each barrier. For this analysis only one type of barrier is considered, namely the Harbeck BigBag system, see Ogunyoye *et al.* (2011) and Big Bag Harbeck GmbH (2014).

The Harbeck BigBag barrier is made from wooden frames and specially coated woven geo-textile bags filled with sand and it is 0.75m high. It is possible to stack up to three bags on top of each other, for a maximal protection height of 2.25m. The barrier system does not necessitate previous installation of fundament or anchoring system and is fast in its deployment. According to Ogunyoye *et al.* (2011), it is unlikely to breach and it is recommended for use on several terrain types and as a second line of defence away from the water course. Figure 7.4 illustrates an individual element of the Harbeck BigBag system, as well a barrier in use during a flood event.

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<sup>12</sup> All currencies from literature are translated into Swiss Francs (CHF), with September 2014 exchange rates.



Figure 7.6 - On top one Harbeck BigBag barrier element, at the bottom Harbeck BigBag local flood barrier deployed (all pictures Copyright Harbeck GmbH)

In the present application, it is considered that local flood barriers consist of three stacked layers of Harbeck BigBag, i.e. the protection height is  $h_{barrier} = 2.25m$ . Given the flexibility of this barrier system, in practice, the decision maker could choose the barrier locations almost freely. However, it is here considered that the local flood barriers are placed on borders of grid cells as illustrated in Figure 7.7. The considered grid cells are square with  $100m$  side length; therefore,  $400m$  of Harbeck BigBag barrier are necessary to protect one grid cell. A building is considered to be protected by a local flood barrier when its centroid lies within the protected area.

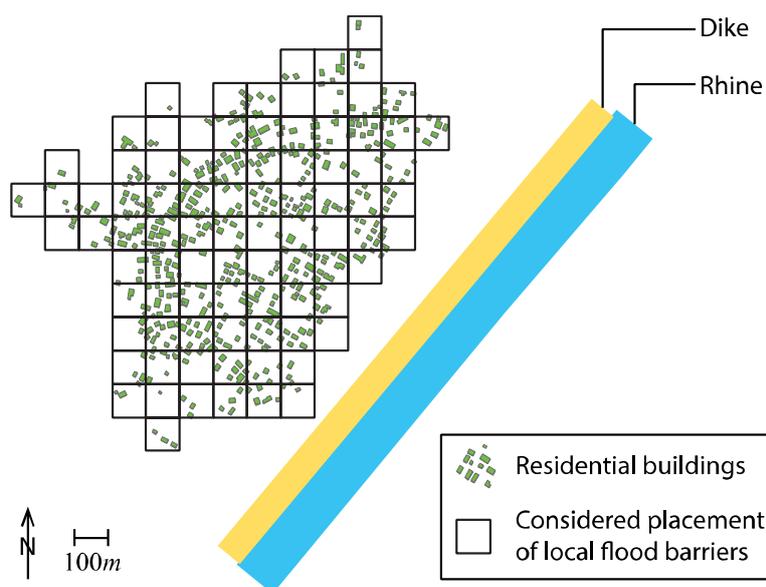


Figure 7.7 - Grid cells on which borders the deployment of local flood barrier is considered

For each grid cell two decision alternatives are considered in the decision analysis:

- Do not deploy a local flood barrier.
- Deploy a local flood barrier.

The number of decision alternatives for the whole second hierarchy level entails all possible combinations of decisions for individual grid cells. The set of decision alternatives is  $A_2 = \{a_2^{(0)}, a_2^{(1)}, \dots, a_2^{(K_2)}\}$  and includes  $K_2 = 2^{N_{LFB}}$  decision alternatives, where  $N_{LFB} = 89$  is the number of considered grid cells. However, decisions are taken for each grid cell individually, resulting in  $N_{LFB}$  binary decisions rather than one decision with  $K_2 + 1$  decision alternatives.

The cost for 100m of barrier is 31'200 CHF (Ogunyoye *et al.* 2011), the protection of a grid cell therefore costs 124'800 CHF. The cost of each decision alternative is found by multiplying the number of protected grid cells by the cost of protecting one grid cell. If two or more contiguous grid cells are protected, it is considered that the shared cell side is not protected twice. The life time of a local flood barrier is assumed to be  $T_2 = 50$  years. Furthermore, it is assumed that the deployment time of local flood barriers is never an issue.

#### 7.2.4.3 Building envelope

The third hierarchy level is formed by building envelopes, considered as a last protection for building interiors. Protection provided by the building envelope might be improved by applying dry-proofing measures, i.e. by making the building envelope watertight to reduce water infiltration into the building interior. Dry-proofing of a building generally involves a number of individual measures, which together effectively reduce water infiltration. A detailed overview of measures to dry-proof a building is found, e.g. in Wingfield *et al.* (2005). In this example, dry-proofing of a residential building is considered to entail the measures listed in Table 7.2. As described in Section 7.3.2.3, the effect of these measures on water infiltration is captured by adapting the building parameterisation in the vulnerability model.

Dry-proofing measures are assumed to be implemented up to 1m over ground floor level. According to several sources, e.g. Kelman (2002), dry-proofing above this threshold significantly increases the failure probability of the building envelope due to hydrostatic pressure differential and it is therefore not recommended.

A decision on dry-proofing is taken for each building individually. As such, for each building the following decision alternatives are available:

- Do not dry-proof the building.
- Dry-proof the building.

Given the number of buildings in the study area  $N_r = 480$ , the set  $A_3 = \{a_3^{(0)}, a_3^{(1)}, \dots, a_3^{(K_3)}\}$  includes  $K_3 = 2^{N_r}$  decision alternatives. However, similarly to local flood barriers, an independent decision is taken for each individual building, reducing the complexity of the decision problem.

Table 7.2 - List of measures required to dry proof a masonry building

Measure	Description
Sprayed on cement	The external building walls are sealed with spray-on cement. This reduces water infiltration through wall cracks.
Flood resistant external door or flood gate	Normal doors, especially of a certain age, tend to have large door cracks through which water can infiltrate the building. Here two options are available for flood proofing the door: 1) install a water- proofed door, or 2) install a flood gate in front of the door.
Airbrick covers	Air bricks are used to ventilate the building and the cavity between building sheets in cavity walls; they can be sealed with simple covers.
Non-return valves in waste pipes and outlets	When sewer system is at capacity and flood water pressure is increasing, water can flow backwards through sanitary installations into the building. A backflow valve blocks waste water from flowing into the building.
Wall openings (e.g. windows and doors) are reinforced	Wall openings, in particular windows, may fail due to hydrostatic and hydrodynamic pressure and a failure may lead to a rapid flooding of the building. Such failures can be avoided through appropriate reinforcement of wall openings.

Indications from literature on the costs for dry-proofing of a building differ (Gersonius *et al.* 2008, Kreibich *et al.* 2011, Poussin *et al.* 2012, Joseph 2014), ranging from 3'000 CHF (Gersonius *et al.* 2008) to over 24'000 CHF (Kreibich *et al.* 2011). In this analysis, dry-proofing costs for a building are assumed to be proportional to the building perimeter. A cost of 200 CHF /m is postulated, which results in dry-proofing costs per building between 8'000 CHF and 15'000 CHF, comparable to Zevenbergen *et*

al. (2007) and Joseph (2014). The life time of dry-proofing measures is assumed to be  $T_3 = 25 \text{ years}$ .

#### 7.2.4.4 Decision alternatives for the hierarchical flood protection system

Given the set of decision alternatives  $A_1, A_2, A_3$  for each considered hierarchy level, the set of decision alternatives available for the whole flood protection system is:

$$A_s = \{a_s^{(k_1, k_2, k_3)}, k_1 = 1, 2, 3; k_2 = 1, 2, \dots, K_2; k_3 = 1, 2, \dots, K_3\}, \quad (7.1)$$

where  $a_s^{(k_1, k_2, k_3)} = (a_1^{(k_1)}, a_2^{(k_2)}, a_3^{(k_3)})$ .

### 7.3 Flood risk assessment

Flood risk is assessed according to the general methodology for hierarchical flood protection systems outlined in Section 4.3, with several simplifications:

- Given a hazard event and dike failure scenario, hazard is modelled in a deterministic manner; this means that the conditional probability density functions  $f_{\mathbf{x}_0|hz_m}(\mathbf{x}_0|hz_m; \Phi_{\mathbf{x}_0})$  and  $f_{\mathbf{x}_i|\mathbf{x}_{i-1}, S_i}(\mathbf{x}_i|\mathbf{x}_{i-1}, S_i; \Phi_{\mathbf{x}_i}), i = 1, \dots, N$  introduced in Chapter 4 are degenerate.
- The breaching of local flood barriers is not considered, i.e. the area protected by a local flood barrier is only considered to be flooded when the barrier is overtopped.
- As described in Chapter 6, the state of the building envelope and water depth inside the building are considered to be dependent. As such, the assumption of conditional independence between the state of a protection structure and the hazard downstream of the structure does not hold.

Figure 7.8 illustrates the methodology introduced in Section 4.3 and its implementation in the present example.

	Variable(s)	Flood risk assessment methodology (Section 4.3)	Example application	
			Adapted methodology	Implementation
	$\mathbf{X}_0$	$h_{z_m} \in \Omega_{h_{z_m}}$	$h_{z_m} \in \mathbf{HZ}$	Flood event $h_{z_m}$ is defined through a synthetic hydrograph
	$S_1$	$f_{\mathbf{x}_0 h_{z_m}}(\mathbf{x}_0   h_{z_m}, \Phi_{\mathbf{x}_0})$	$\mathbf{x}_0 = g_0(h_{z_m}, \Phi_{\mathbf{x}_0})$	Flow routing model (LISFLOOD-FP)
	$\mathbf{X}_1$	$f_{S_1 \mathbf{x}_0} (s_1   \mathbf{x}_0; \Phi_{S_1})$	$f_{S_1 \mathbf{x}_0} (s_1   \mathbf{x}_0; \Phi_{S_1})$	Dike fragility model
	$S_2$	$f_{\mathbf{x}_1 \mathbf{x}_0, S_1} (\mathbf{x}_1   \mathbf{x}_0, s_1; \Phi_{\mathbf{x}_1})$	$\mathbf{x}_1 = g_1(\mathbf{x}_0, s_1, \Phi_{\mathbf{x}_1})$	Flow routing model (LISFLOOD-FP)
	$\mathbf{X}_2$	$f_{S_2 \mathbf{x}_1} (s_2   \mathbf{x}_1; \Phi_{S_2})$	$S_2 = constant$	Breaching of local flood barrier is not considered
	$S_3$	$f_{\mathbf{x}_2 \mathbf{x}_1, S_2} (\mathbf{x}_2   \mathbf{x}_1, s_2; \Phi_{\mathbf{x}_2})$	$\mathbf{x}_2 = g_2(\mathbf{x}_1, s_2, \Phi_{\mathbf{x}_2})$	Indicator function
	$\mathbf{X}_3$	$f_{S_3 \mathbf{x}_2} (s_3   \mathbf{x}_2; \Phi_{S_3})$	$f_{\mathbf{x}_3, S_3 \mathbf{x}_2} (\mathbf{x}_3, s_3   \mathbf{x}_2; a_3^{(b)})$	Vulnerability model for residential buildings (Chapter 6)
	$L_j$	$f_{L_j \mathbf{x}_3, S_3} (l_j   \mathbf{x}_3, s_3; \Phi_{L_j})$		

Figure 7.8 - Modelling overview

In this example, water depth  $h_w$  is considered as the only hazard index. As such,  $\mathbf{x}_0$  contains realisations of water depth for the whole river channel. Similarly,  $\mathbf{x}_1$  contains realisations of water depth for the whole flood plain between the dike and any local flood barrier;  $\mathbf{x}_2$  contains realisations of water depth in the area protected by local flood barriers and  $\mathbf{x}_3$  contains realisations of water depth inside buildings. As  $h_w$  is the only considered hazard index, dike breaching, local flood barrier overtopping and monetary loss to buildings are assessed in function of, or conditional on,  $h_w$  impacting the protection structure or building. In the following, the notation  $\mathbf{x}_i, i = 0, 1, 2, 3$  is utilised for water depth across whole areas (e.g. the whole river channel or the whole flood plain) and the notation  $h_w$  is utilised for water depth impacting a particular protection structure or building.

Categorical variable  $S_1$  indicates the state of the dike. Breaching of local flood barrier is neglected, therefore, the state  $S_2$  of local flood barrier is considered constant. The state  $S_3$  of building envelopes is modelled with the vulnerability model described in Chapter 6.

The parameter vectors  $\boldsymbol{\varphi}_{\mathbf{x}_0}$  and  $\boldsymbol{\varphi}_{\mathbf{x}_1}$  contain the parameterisation of the flow routing model, including the digital elevation model of the study area (Swisstopo 2005), surface roughness parameter (Manning's  $n$ ), and river geometry. Parameter vector  $\boldsymbol{\varphi}_{S_1}$  contains dike height  $h_{dike}$  as well as parameterisation of the dike fragility model (as described in Section 7.3.2.1). Parameter vector  $\boldsymbol{\varphi}_{S_2}$  contains height  $h_{barrier}$  and location of local flood barriers. Parameter vector  $\boldsymbol{\varphi}_{L_j}, j = 1, 2, \dots, N_r$  contains all parameters to assess monetary loss to building  $j$  in accordance with the vulnerability model described in Chapter 6. As such, parameter vector  $\boldsymbol{\varphi}_{S_3}$  is a collection of all  $\boldsymbol{\varphi}_{L_j}, j = 1, \dots, N_r$ .

In the right column of Figure 7.8, the implementation approach of all modelling steps is given; each is described in more detail in the following sections, however, a comment on hazard modelling is given here. According to the general methodology, hazard is modelled in steps, with each step delimited by the protection structures at two hierarchy levels. As previously mentioned, this would require a lumped flow routing model to be employed; however, to allow an accurate modelling of hazard a distributed flow routing model is chosen and thus a different implementation approach is identified. Hazards in the river channel and on the flood plain are jointly assessed conditionally on the state  $S_1$  of the dike and without consideration of any local flood barriers. From the results of the flow routing model, the functions  $\mathbf{x}_0 = g_0(hz_m, \boldsymbol{\varphi}_{\mathbf{x}_0})$  and

$\mathbf{x}_1 = g_1(\mathbf{x}_0, s_1, \boldsymbol{\varphi}_{\mathbf{x}_1})$  can be derived if necessary. The impact of local flood barriers on hazard is modelled by modifying the results of the flood routing model.

### 7.3.1 Receptors

Receptors considered in this application are  $N_r = 480$  buildings, identified with subindex  $j = 1, \dots, N_r$ . Each building  $j$  is characterised by parameter vector  $\boldsymbol{\varphi}_{L_j}$ , which not only parametrises building vulnerability in accordance with Chapter 6, but also its location. As such,  $\boldsymbol{\varphi}_{L_j}$  includes all information necessary to determine monetary loss to building  $j$ .

Data on location and floor area of each building is available from Swisstopo (2010). Information on the number of stories is available as an aggregated statistics from Bundesamt für Statistik (2014); it indicates that 90% of single-family residential buildings in Canton St. Gallen are 2-storey. No data is available in regard to other building characteristics. It is therefore postulated that the main building material for all buildings is masonry and that 50% of buildings have a basement. Storey heights are assumed to be normally distributed with mean  $3m$  and standard deviation  $0.5m$ . The building value is modelled in function of its volume; the monetary value per cubic meter is assumed to be  $2000 \text{ CHF}/m^3$ .

Each building is characterised with available knowledge and by sampling from aggregated statistics. Given the lack of data, epistemic uncertainty on building configuration and its monetary value is large, prompting a building parameterisation reflecting large uncertainty. However, the choice is made to parameterise buildings with little uncertainty. The reason for this modelling choice is to replicate the conditions under which a decision maker would optimise a hierarchical flood protection system in practice. At the scale of this model, it is reasonable to assume that the decision maker would have detailed data on each building; hence the modelling choice is taken to utilise the parameterisation for “Known building configuration” described in Chapter 6.

### 7.3.2 Fragility and vulnerability models

Fragility models for dike and local flood barriers, as well as a building vulnerability model are introduced.

#### 7.3.2.1 Dike fragility

The dike fragility model allows for modelling breaching of the dike. In a common modelling approach, see e.g. Hall, Dawson *et al.* (2003), the dike is subdivided into dike sections. In particular, the 6km dike here considered is divided into  $N_s = 10$  dike sections of 600m length as illustrated in Figure 7.9. According to several sources, e.g. Vorogushyn *et al.* (2010), for dike sections of this length, the structural response of individual dike sections can be assumed to be conditionally independent given the load from a hazard event. A breach is understood as structural failure of a dike section, leading to a removal of the dike cross-section.

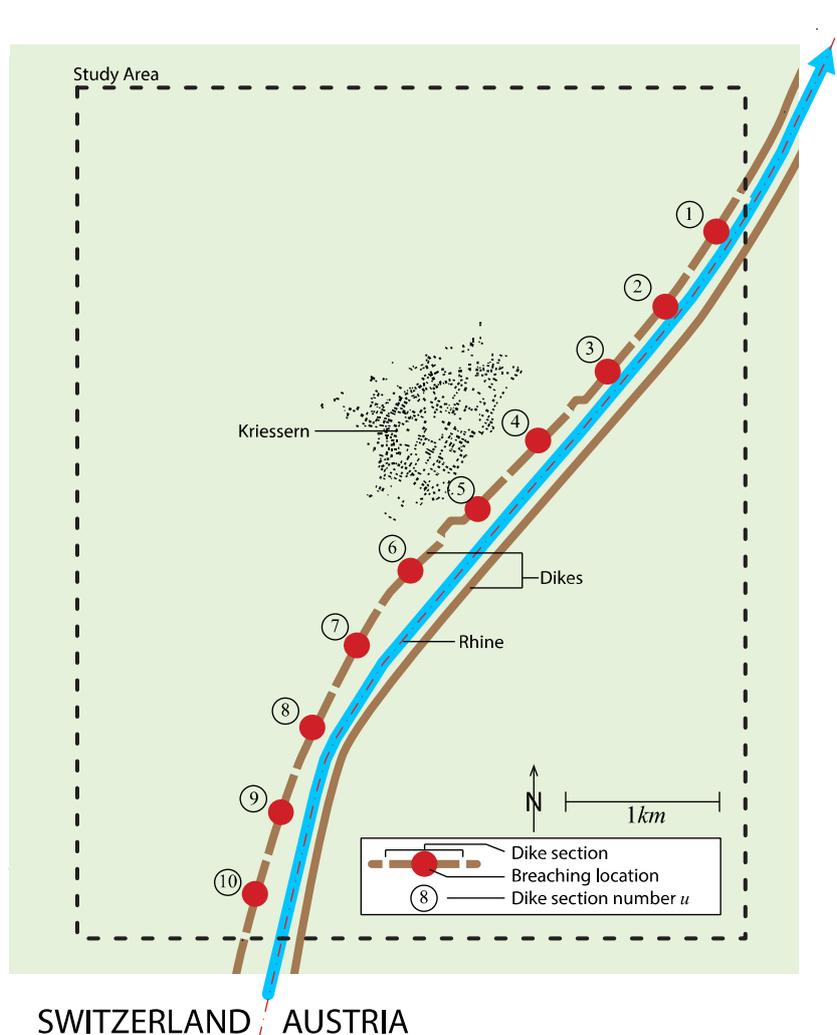


Figure 7.9 - Considered dike sections and breaching locations

In the present analysis, no information on dike geometry and construction material is available. Therefore, the fragility of dike sections cannot be modelled in detail and a simplified approach is taken instead. A fragility curve is postulated for each considered dike decision alternative in set  $A_1$ . Each fragility model defines the breaching probability for dike sections  $u=1, \dots, N_s = 10$  in function of the water depth  $h_w^{(u)}$  from the foot of the dike (see Figure 7.10). As  $h_w^{(u)}$  is the water depth within the river channel, it is an element of  $\mathbf{x}_0$ .

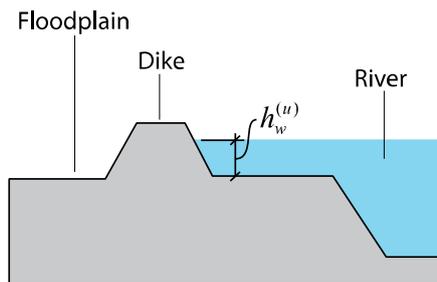


Figure 7.10 - Water depth  $h_w^{(u)}$  from the foot of the dike used to calculate the dike breaching probability

Defining random event  $B^{(u)}$  as a breach of the  $u$  th dike section, the probability  $P[B^{(u)} | h_w^{(u)}]$  is assumed to follow the lognormal distribution function. That is:

$$P[B^{(u)} | h_w^{(u)}] = \Phi\left(\frac{\ln(h_w^{(u)}) - \mu}{\sigma}\right) \quad (7.2)$$

where  $\Phi(\cdot)$  is the standard normal distribution function,  $\sigma$  is the scale parameter of the lognormal distribution and  $\mu$  is its log-shape parameter. Parameter values for each dike decision alternatives are provided in the table below. The resulting fragilities curves are illustrated in Figure 7.11.

Table 7.3 - Parameter values for each dike decision alternatives

Decision alternative	Dike height over floodplain	Distribution parameters
Current state - $a_1^{(0)}$	3 m	$\mu = 1, \sigma = 0.7$
Alternative 1 - $a_1^{(1)}$	3 m	$\mu = 1.4, \sigma = 0.6$
Alternative 2 - $a_1^{(2)}$	5 m	$\mu = 1.7, \sigma = 0.4$

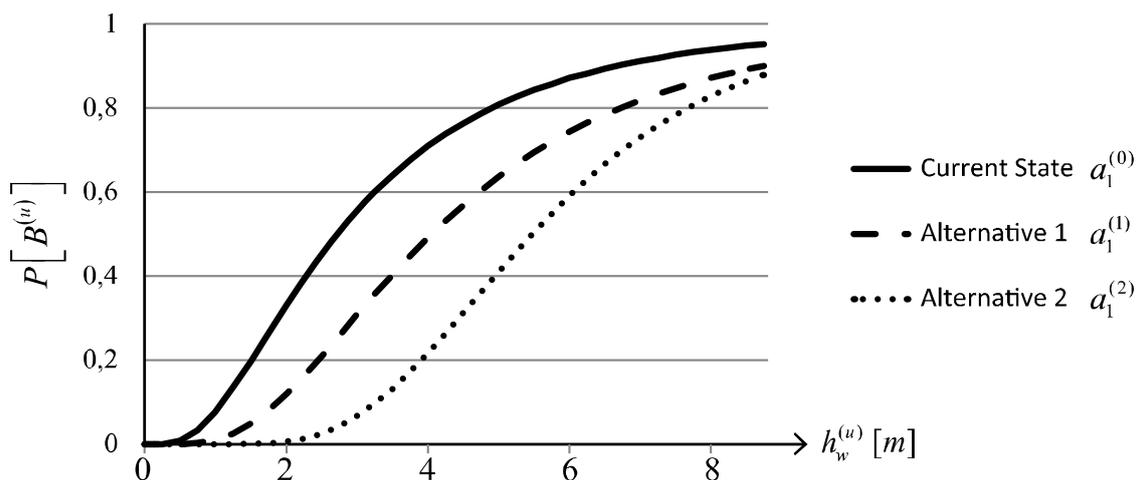


Figure 7.11 - Postulated fragility curve for each dike decision alternative

### 7.3.2.2 Fragility of local flood barrier

According to Ogunyoye *et al.* (2011), likely failure modes for the Harbeck BigBag barrier system are overtopping, overturning, sliding and seepage. Conversely, collapse, rolling and breach of the barrier are deemed unlikely.

Overtopping and sliding of the barrier can be avoided by stabilising the barrier with a support structure on its dry side. Seepage is deemed to be smaller than 40 litres per hour and meter of installed barrier. As such, it seems that, if the barrier is correctly installed, only overtopping is a significant failure mode and no further fragility model is necessary.

### 7.3.2.3 Vulnerability of residential buildings

The vulnerability model introduced in Chapter 6 is utilised to model vulnerability of residential buildings, allowing representation of the impact of dry-proofing measures on vulnerability. Each building in the study area is modelled individually to allow for detailed consideration of its characteristics. As mentioned in Section 7.3.1, the  $j$ th building is characterised by parameter vector  $\phi_{L_j}$ . The parameterisation of building without dry-proofing follows the description in Chapter 6. From it, the parameterisation of the building with dry-proofing is derived by modifying parameters to reproduce the impact of dry-proofing measures described in Table 7.2. To identify which parameters to modify, heuristics are formulated describing the impact of individual dry-proofing measures on damage processes. Then, building and component parameters are identified which allow reproducing the heuristic in the model, see Table 7.4.

Table 7.4 - Heuristic effect of flood proofing measure on building vulnerability and corresponding change in parameterisation (the parameters are defined in Chapter 6)

Flood proofing measure	Heuristic	Modified parameters
- Spray-on cement - Air brick cover - Backflow valves	No water infiltration below 1m from the ground floor level.	$A_j   (DM_j = dm_{j0}) = 0$ <sup>13</sup> for all envelope components below 1m from the ground floor level.
- Flood-gate /flood proofed door - Obstruct basement windows	The building components below 1m from the ground floor level are reinforced so that a mechanical failure from water pressure differential becomes unlikely.	- $f_c \gg$ for all envelope components below 1m from the ground floor level <sup>14</sup> . - $f_t \gg$ for all envelope components below 1m from the ground floor level.

An example of resulting vulnerability curves are illustrated in the figure below for a 2-storey building without basement.

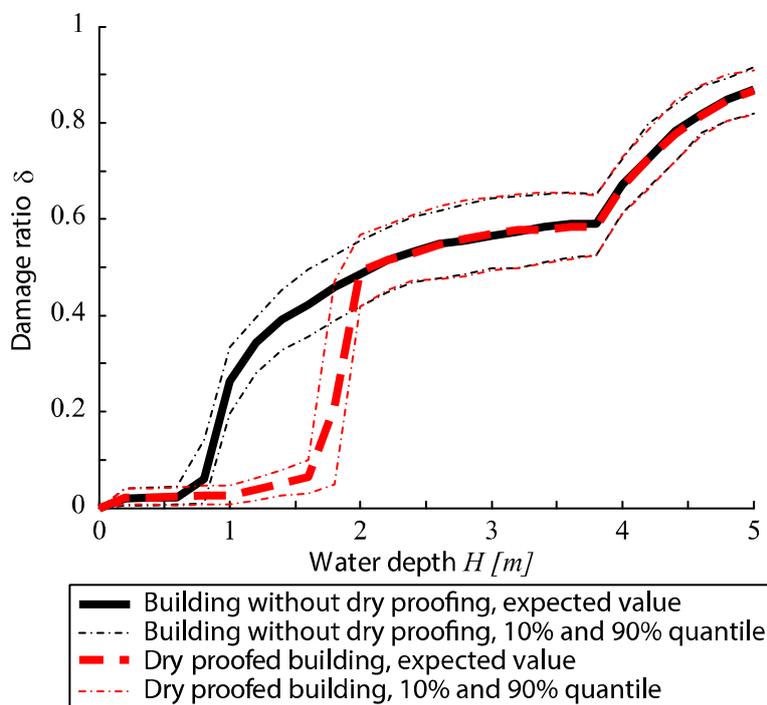


Figure 7.12 - Vulnerability curves for an example 2-storey masonry building without basement with and without dry-proofing measures

<sup>13</sup> For definition of  $A_j | (DM_j = dm_{j0})$  see Section 6.3.2.2 on page 131.

<sup>14</sup> For definition of  $f_c, f_t$  see Figure 6.7 on page 127.

The vulnerability curves in Figure 7.12 illustrate how dry-proofing markedly reduces damage ratios at small water depth, since the water cannot infiltrate the building. When water depth increases, the dry-proofing measures lose their effect and water can enter the building.

The monetary loss  $L_j$  for building  $j$  is modelled (as described in Chapter 6) by multiplying damage ratio with building value.

### 7.3.3 Hazard assessment

The aim of the hazard assessment is to determine the probabilistic characterisation of water depth at each location in the study area. The hazard assessment entails several steps; Figure 7.13 provides an overview.

In a first step the hazard source process (river discharge) is characterised. The probabilistic characterisation of extreme river discharge  $Q$  is modelled through its probability density function  $f_Q(q)$ . The sample space  $\Omega_{HZ}$  of all possible flood events affecting the study area is approximated through an event set  $\mathbf{HZ} = \{hz_1, hz_2, \dots, hz_M\}$ . Each event  $hz_m$  is characterised by a synthetic event hydrograph and a maximum river discharge  $q_{m,\max}$ , which is sampled from  $f_Q(q)$  with a risk-based sampling method derived from Dawson *et al.* (2005). The occurrence probability  $p_{hz_m}$  of the event  $hz_m$  is also determined from  $f_Q(q)$ . River discharges and flood plain discharges are determined with a flow routing model (LISFLOOD-FP, Bates and De Roo 2000) for each hazard event  $hz_m$  and dike breaching scenario (where necessary).

Local flood barriers are not considered in the flow routing model as the impact of local flood barriers on hazard is modelled in a later step. The water depth within buildings is calculated with the vulnerability model described in Chapter 6.

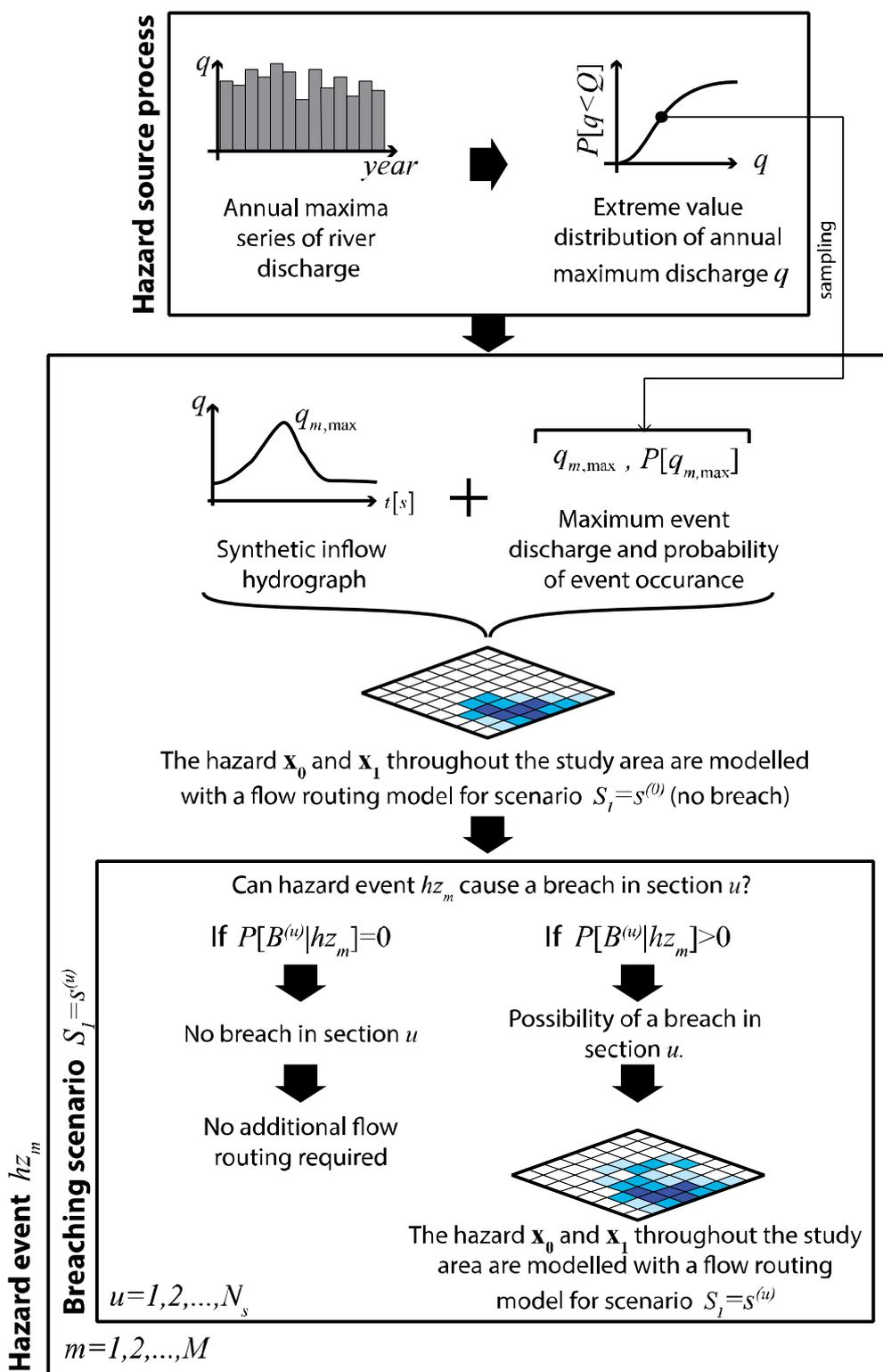


Figure 7.13 - Hazard assessment overview

### 7.3.3.1 Probabilistic characterisation of extreme river discharge

Probabilistic characterisation of extreme river discharge is modelled through analysis of the annual maxima series. Annual maximum discharge data for the gauging station of Diepoldsau is available from BAFU (2014) from 1919 until present. From it, the probability density function of annual maximum discharge  $f_Q(q)$  can be estimated<sup>15</sup>. The probability distribution proposed by BAFU (2014) is utilised to model  $f_Q(q)$ , i.e. a gamma distribution with mean  $1337.48m^3/s$  and standard deviation  $448.87 m^3/s$ . In this example application it is assumed that natural processes underlying floods are stationary; this means that any possible change in future flood risk, e.g. due to climate change, is disregarded.

By modelling the hydrological load through an analysis of annual maxima series, the modelled risk will be on an annual basis and it is implicitly assumed that the probability of two or more events per year is negligible.

### 7.3.3.2 Hydrological event

Hydrological event  $hz_m \in \mathbf{HZ}$  is characterised with a synthetic hydrograph  $q_m(t)$  at the river inflow into the study area. Methods are available to define the shape of synthetic hydrographs from empirical observations, e.g. through cluster analysis of past events, see Apel *et al.* (2006). However, data to perform such an analysis is not available for the present study. Therefore, a synthetic hydrograph is freely postulated. The shape of the hydrograph is illustrated in Figure 7.14. In each event inflow discharge is assumed to start at  $200m^3/s$  and linearly increase to peak discharge  $q_{m,max}$ , stay constant at peak-discharge for approximately 90 minutes and decrease thereafter. Note that the base discharge (normal river discharge before the start of a hazard event) is directly considered in the initial conditions of the flow routing model.

The peak river discharge  $q_{m,max}$  is sampled from  $f_Q(q)$  in such a way, that the whole event set  $\mathbf{HZ}$  allows for a good representation of flood risk in the study area. The sampling methodology is described in Section 7.3.3.6.

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<sup>15</sup> Note that the gauging station of Diepoldsau is located well within the study area, i.e. not at the river inflow into the study area. The catchment areas (which generally correlate with the river discharge) at the river inflow into the study area and at the gauging station of Diepoldsau are similar in size, hence, it is reasonable to utilize the annual maximum discharge data from the gauging station to model the probability distribution of extreme river discharge at the river inflow into the study area.

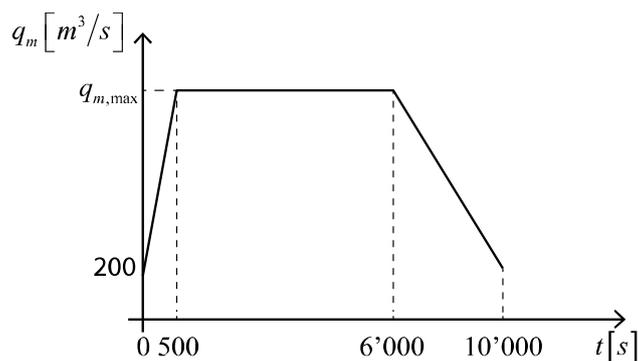


Figure 7.14 - Synthetic hydrograph

### 7.3.3.3 Breaching scenarios

Breaches can in principle, occur at any location along the dike, however breaches are here considered to occur in discrete dike sections  $u=1, \dots, N_s$  (as illustrated in Figure 7.9). The structural response of dike sections is modelled with the fragility model described in Section 7.3.2.1; the responses of individual dike sections are assumed to be conditionally independent given the hydrologic load from flood event  $hz_m$ , i.e. breaching of each section  $u=1, \dots, N_s$  is considered conditionally independent given river water depth  $x_0$ .

In principle more than one dike section can breach during a hazard event, however here it is assumed that only one breach at a time occurs. As such, for each hazard event  $hz_m$ ,  $N_s + 1$  scenarios are considered. Scenarios are identified through dike state variable  $S_1$ : the scenario with no breach is identified as  $S_1 = s^{(0)}$ , the scenario entailing a breach in the  $u$  th dike section,  $u=1, \dots, N_s$ , is identified as  $S_1 = s^{(u)}$ .

The probability  $P[B^{(u)} | h_w^{(u)}]$  of a breach event in section  $u$  is calculated with the dike fragility model formulated in Equation (7.2). From it, the probability of scenarios  $S_1 = s^{(0)}$  and  $S_1 = s^{(u)}$ ,  $u=1, \dots, N_s$  is derived:

- Scenario  $S_1 = s^{(u)}$  entails a breach only in the  $u$  th section and no breach in any other sections. Given that breaches are considered conditionally independent given hazard event  $hz_m$ , the occurrence probability of scenario  $S_1 = s^{(u)}$  is calculated as:

$$P[S_1 = s^{(u)} | hz_m] = P[B^{(u)} | h_w^{(u)}] \prod_{v=1 \setminus u}^{N_s} (1 - P[B^{(v)} | h_w^{(v)}]), \quad (7.3)$$

- The probability of scenario entailing no dike breaches is:

$$P[S_1 = s^{(0)} | hz_m] = 1 - \prod_{u=1}^{N_s} (1 - P[S_1 = s^{(u)} | hz_m]) \quad (7.4)$$

This formulation assumes that probability of scenarios with two or more breaches is negligible.

Whereas the occurrence of breaches is considered probabilistically, its geometrical features are modelled deterministically; based on observed earth dike breaches as reported by Apel *et al.* (2009) and Vorogushyn *et al.* (2010), a rectangular breach with depth  $3m$  and the width  $120m$  is postulated.

#### 7.3.3.4 Flow routing

River channel and flood plain flow routing are modelled with the distributed flow routing model LISFLOOD-FP. Within LISFLOOD-FP, river channel flow routing is modelled with the one-dimensional kinematic wave approximation of the Navier-Stokes equations and floodplain flow routing is modelled for floodplain grid cells as a function of flood plain friction and free surface height difference across each cell face (Bates *et al.* 2013).

LISFLOOD-FP is run for each considered hazard event  $hz_m \in \mathbf{HZ}$  and scenario  $S_1 = s^{(0)}$ . The results of the model runs includes the water depth  $h_w^{(u)}$  impacting dike section  $u = 1, \dots, N_s$ ; the probability of breach  $P\left[B^{(u)} \mid h_w^{(u)}\right]$  can thus be assessed for each dike section with the dike fragility model. LISFLOOD-FP is run again to model scenario  $S_1 = s^{(u)}$  for all  $u = 1, \dots, N_s$  where  $P\left[B^{(u)} \mid h_w^{(u)}\right] > 0$ .

The dike geometry is directly modelled within LISFLOOD-FP and dike overtopping is automatically considered with the standard weir equations see e.g. Chadwick *et al.* (2004). However, dike breach scenarios  $S_1 = s^{(u)}, u = 1, 2, \dots, N_s$  need to be given special consideration, by modifying the dike geometry in LISFLOOD-FP as described in the following. At start of scenario  $S_1 = s^{(u)}$  the dike is assumed to be intact. When river water depth  $h_w^{(u)}$  reaches its maximal value, LISFLOOD-FP is stopped and the intermediate results stored. A breach is then inserted in the dike geometry of section  $u$ , and the flow routing model is restarted from the stored intermediate results.

The output of LISFLOOD-FP includes a time series of water depth for each river cross-section and each floodplain grid cell in the study area. As previously mentioned only the maximal water depth  $h_w$  throughout the study area is considered as a hazard index and stored in hazard vectors  $\mathbf{x}_0$  and  $\mathbf{x}_1$  for all hazard events  $hz_m \in \mathbf{HZ}$  and breaching scenarios  $S_1 = s^{(u)}, u = 0, 1, \dots, N_s$ . The stored flow routing results would allow for extrapolating functions  $\mathbf{x}_0 = g_0(hz_m, \boldsymbol{\varphi}_{\mathbf{x}_0})$  and  $\mathbf{x}_1 = g_1(\mathbf{x}_0, s_1, \boldsymbol{\varphi}_{\mathbf{x}_1})$  in accordance

with the flood risk assessment methodology described in Chapter 4. However, the explicit extrapolation is not necessary for this example application.

The impact of local flood barriers on flow routing is disregarded, i.e. it is assumed that local flood barriers are too small to significantly affect the floodplain water flow. However, the impact of local flood barriers on hazard characteristics is considered, as described in the following paragraph.

### 7.3.3.5 Hazard considering local flood barriers

For areas protected by a local flood barrier, hazard within flood barriers is described with function  $\mathbf{x}_2 = g_2(\mathbf{x}_1, \boldsymbol{\varphi}_{S_2})$ . As detailed in the Section 7.3.2.2, breaching of local flood barriers is not considered and a protected grid cell is only flooded when its barrier is overtopped.

Overtopping occurs when the water depth  $h_w$  impacting the barrier exceeds the barrier height  $h_{barrier}$ . Although initial water flow into a protected area may be small, it is assumed a barrier loses its functionality when overtopped. As a consequences, if  $h_w$  exceeds  $h_{barrier}$ , water depth within the protected area is assumed to become what it would have been, had there been no local flood barrier, i.e:

$$\mathbf{x}_2 = g_2(\mathbf{x}_1, \boldsymbol{\varphi}_{S_2}) = I[h_w > h_{barrier}] \cdot \mathbf{x}_1 \quad (7.5)$$

where  $I[\cdot]$  is an indicator function which returns 1 when the condition in brackets is satisfied and 0 otherwise,  $h_w \in \mathbf{x}_1$  is the water depth impacting the local flood barrier and  $h_{barrier} \in \boldsymbol{\varphi}_{S_2}$  is the local flood barrier height.

### 7.3.3.6 Event sampling

For each event  $hz_m \in \mathbf{HZ}$ , peak discharge  $q_{m,max}$  is sampled from the probability density function  $f_Q(q)$  of annual maximum discharge. The sampled events must be chosen so that the event set  $\mathbf{HZ}$  gives a good approximation of flood risk in the study area. Due to computational costs of flow routing, a crude Monte Carlo sampling approach is infeasible. Instead a risk-based sampling approach derived from Dawson *et al.* (2005) is utilised. The approach is first introduced in general terms and detailed thereafter.

A small number  $M^{(0)}$  of peak event discharges  $q_{m,max}, m = 1, \dots, M^{(0)}$  is sampled in regular intervals from  $f_Q(q)$ . The initial sample range is chosen so that all discharge values that could possibly contribute to flood risk are within the range. For each of these initial events, an approximated risk measure is calculated. This gives a first approximate

distribution of risk in function of peak event discharge  $Q$ . It then becomes clear which values of  $Q$  make the largest contribution to risk. Thereafter, additional sampling rounds are undertaken, and in each round, a sample is added to the sample set at a value of  $Q$ , which is expected to make a large contribution to risk. The size of the sample set  $M^{(\lambda)}$ , where  $\lambda$  is the sampling round, is thus increased, until convergence of total risk is reached. The detailed procedure is as follows:

- 1) A first event set **HZ** with  $M^{(0)} = 10$  samples is created. Maximal discharge value  $q_{m,\max}$  for event  $hz_m, m = 1, \dots, M^{(0)}$  is sampled from  $f_Q(q)$  in regular intervals. The range of sampling is chosen such that the smallest event cannot produce any flooding and the largest has a negligible probability of occurrence.
- 2) The probability of occurrence  $p_{hz_m}^{(\lambda)}$  of each event  $hz_m$  is calculated as:

$$p_{hz_m}^{(\lambda)} = \begin{cases} F_Q\left(\frac{q_{m+1,\max} + q_{m,\max}}{2}\right) - F_Q(q_{m,\max}), & m = 1 \\ F_Q\left(\frac{q_{m+1,\max} + q_{m,\max}}{2}\right) - F_Q\left(\frac{q_{m,\max} + q_{m-1,\max}}{2}\right), & m = 2, \dots, M^{(\lambda)} - 1, \\ 1 - F_Q\left(\frac{q_{m,\max} + q_{m-1,\max}}{2}\right), & m = M^{(\lambda)} \end{cases} \quad (7.6)$$

where  $F_Q(q)$  is the cumulative distribution function of the annual maximum discharge. Note that  $p_{hz_m}^{(\lambda)}$  may change with each sampling round, since it depends on the event sampling density.

- 3) Each event is modelled in the flow routing model for all considered scenarios,  $S_1 = s^{(u)}, u = 0, \dots, 10$ , with  $P[S_1 = s^{(u)} | hz_m] > 0$ .
- 4) Given the flow routing results for event  $hz_m$  and scenario  $S_1 = s^{(u)}$ , the approximated scenario consequences  $d_{m,u}$  are estimated with an approximated damage function and the database of building location. The approximated damage function assumes damages to be proportional to the flooded building floor area.
- 5) The risk integral for event  $hz_m$  is approximated with

$$\tilde{r}_m^{(\lambda)} = p_{hz_m}^{(\lambda)} \sum_{u=0}^{N_s} d_{m,u} P[S_1 = s^{(u)} | hz_m], \quad (7.7)$$

and the total risk measure after the  $\lambda$  th sampling round is:

$$\tilde{r}_{tot}^{(\lambda)} = \sum_{m=1}^{M^{(\lambda)}} \tilde{r}_m^{(\lambda)} . \quad (7.8)$$

6) The convergence of  $\tilde{r}_{tot}^{(\lambda)}$  is checked as:

$$\left| \frac{\tilde{r}_{tot}^{(\lambda)} - \tilde{r}_{tot}^{(\lambda-1)}}{\tilde{r}_{tot}^{(\lambda)}} \right| < 1\% \quad (7.9)$$

7) If the convergence criteria is not met and additional sampling round  $(\lambda + 1)$  is started by sampling an additional event from  $f_Q(q)$ . The discharge for the new event is sampled as a neighbour of the event that had the largest value  $\tilde{r}_m^{(\lambda)}$ . Thereafter the procedure from 2) to 6) is repeated.

Note that the above procedure is only employed for the dike decision alternative  $a_1^{(0)}$  and without considering further flood protection structures. For all other configurations of the flood protection system, the same event set **HZ** is utilised.

#### 7.3.4 Flood risk

Annual risk  $r_j$  for building  $j=1, \dots, N_r$  (which is equivalent to the annual expected flood loss) can be calculated as:

$$r_j = \sum_{\mathbf{HZ}} \left( p_{hz_m} \cdot \sum_{u=0}^{N_s} \left( P[S_1 = s^{(u)} | hz_m] \cdot E[L_j | hz_m, S_1 = s^{(u)}] \right) \right), \quad (7.10)$$

where  $p_{hz_m}$  is calculated according to Equation (7.6) and  $E[L_j | hz_m, S_1 = s^{(u)}]$  is the vulnerability model result given hazard event and breaching scenario. Total risk  $r_{tot}$  for all receptors in the study area is:

$$r_{tot} = \sum_{j=1}^{N_r} r_j . \quad (7.11)$$

Annual expected loss is assumed to be stationary over time, i.e. the hazard and the receptor characteristics are assumed to be constant in time.

#### 7.4 Decision model

The flood risk assessment described in Section 7.3 is carried out for all configurations of the hierarchical flood protection system. As such, total risk  $r_{tot}(a_s^{(k_1, k_2, k_3)})$  is a function of decision alternative  $a_s^{(k_1, k_2, k_3)}$ .

##### 7.4.1 Expected utility

The expected utility of decision alternative  $a_s^{(k_1, k_2, k_3)}$  is defined in function of risk reduction,  $\Delta r(a_s^{(k_1, k_2, k_3)})$ , and costs  $c_s^{(k_1, k_2, k_3)}(t)$  it entails. Risk reduction for  $a_s^{(k_1, k_2, k_3)}$  is calculated as:

$$\Delta r(a_s^{(k_1, k_2, k_3)}) = r_{tot}(a_s^{(0,0,0)}) - r_{tot}(a_s^{(k_1, k_2, k_3)}) \quad (7.12)$$

Expected utility is calculated over  $T=100$  years which corresponds to the expected dike life time. It is assumed that, in the considered time frame, local flood barriers and flood proofing measures are renewed according to their respective life times. The construction cost for decision alternatives are captured in  $c_s^{(k_1, k_2, k_3)}(t)$ . The expected utility of  $a_s^{(k_1, k_2, k_3)}$  is then:

$$E[U(a_s^{(k_1, k_2, k_3)})] = \sum_{t=0}^T \frac{1}{(1+\rho)^t} (\Delta r(a_s^{(k_1, k_2, k_3)}) - c_s^{(k_1, k_2, k_3)}(t)) \quad (7.13)$$

where  $\rho = 4\%$  is the discounting rate.

##### 7.4.2 Optimal configuration of the flood protection system

The optimal configuration  $a_s^*$  of the hierarchical flood protection system is found by identifying the decision alternative with the largest expected utility, i.e.:

$$E[U(a_s^*)] = \max_{A_s} E[U(a_s^{(k_1, k_2, k_3)})] \quad (7.14)$$

## 7.5 Results

The results of this example application are presented as follows: first, the effect of hierarchical flood protection system on flood risk is analysed by comparing results for several postulated protection system configurations, with the aim of illustrating the impact on flood risk of different protection hierarchy levels. Then, the decision optimisation result is presented. The value of hierarchical flood protection systems is illustrated through a comparison of optimal system configurations under a number of different decision constraints. Results are described in the present section and discussed in Section 7.6.

### 7.5.1 Risk reduction

A common way of representing natural hazard risk is an exceedance probability curve (EP-curve), which captures the annual probability that a certain loss  $L_{tot} = l_{tot}$  is exceeded. In a first step, an analysis of the risk reduction potential of hierarchical flood protection system is presented. Figure 7.15 illustrates an EP-curve for 12 different flood protection system configurations. These system configurations combine one of the three dike decision alternatives with minimal (no protection) and maximal protection available on the second and third hierarchy levels (i.e. all local flood barriers deployed respectively all buildings dry-proofed). As such, they are indicative for the risk reduction potential available at each hierarchy level and combination thereof. The considered system configurations are (for  $k_1 = 0, 1, 2$ ):

- Only dike ( $a_s^{(k_1, 0, 0)}$ ).
- Dike and all local flood barriers ( $a_s^{(k_1, N_2, 0)}$ ).
- Dike and dry-proofing for all buildings ( $a_s^{(k_1, 0, N_3)}$ ).
- Dike, all local flood barriers and dry-proofing for all buildings ( $a_s^{(k_1, N_2, N_3)}$ ).

The EP-curves for these system configurations are illustrated in Figure 7.15.

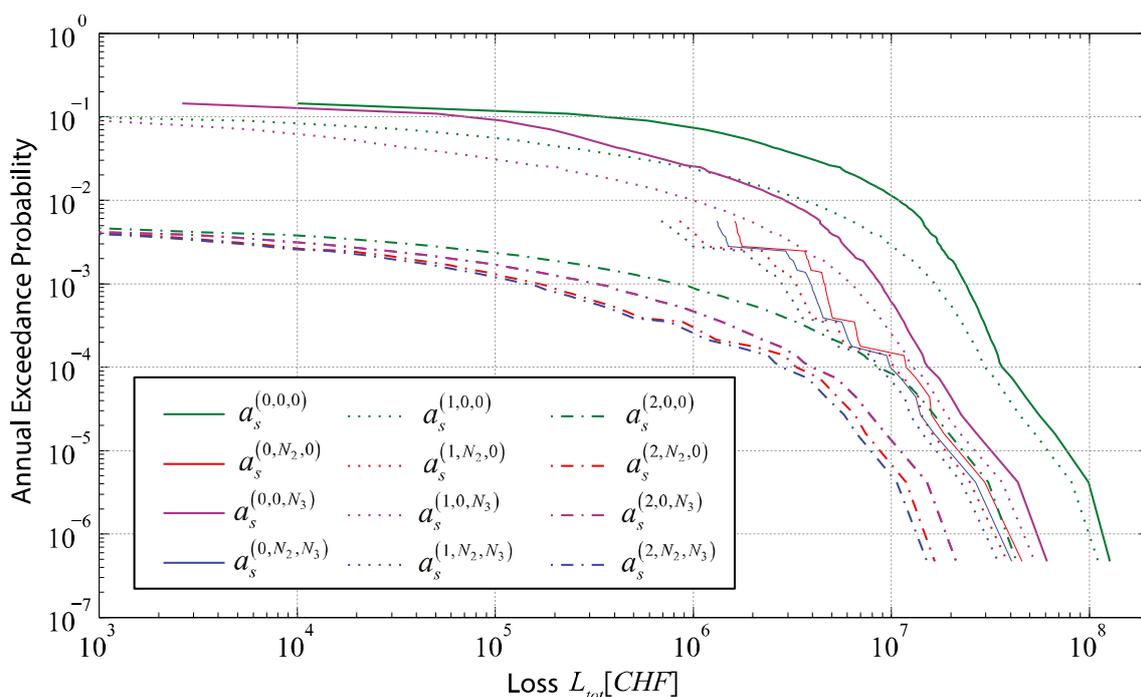


Figure 7.15 - Annual exceedance probability of loss  $L_{tot}$  for different configurations of the protection system

From Figure 7.15 the following observations are made. The current system configuration  $a_s^{(0,0,0)}$  entails the largest flood risk. Compared to the current dike, a strengthened dike ( $a_s^{(1,0,0)}$ ) reduces risk slightly; a heightening and strengthening of the dike ( $a_s^{(2,0,0)}$ ) reduces risk markedly. The addition of further protection levels effectively reduces risk entailed in each dike decision alternative. However, the marginal risk reduction from adding a protection level seems to decrease with increasing number of protection levels. Dry-proofing (decision alternatives  $a_s^{(k_1,0,N_3)}$ ,  $k_1 = 0,1,2$ ) seems to reduce size of losses, but does not affect the probability of incurring a loss. Conversely, local flood barriers (decision alternatives  $a_s^{(k_1,N_2,0)}$ ,  $k_1 = 0,1,2$ ) reduce loss size, as well as the probability of incurring a loss. The combination of dry-proofing and local flood barriers is only marginally better than the system configuration, comprising only local flood barriers.

### 7.5.2 Optimised flood protection system

The hierarchical flood protection system for Kriessern is optimisation to maximise expected utility according to Equation (7.14).

To illustrate the value of hierarchical optimisation of a hierarchical flood protection system, additional decision analyses are carried out; in each, the set of available decision alternatives is constrained to flood protection system configurations with one

or two protection levels. A total of four decision analyses are carried out, with the following constraints:

- Decision optimisation SSO – Single-structure optimisation, only the dike is considered. The set of available decision alternatives is  $A_{s,SSO} = \{a_s^{(1,0,0)}, a_s^{(2,0,0)}, a_s^{(3,0,0)}\}$ .
- Decision optimisation HO1 – Two-level hierarchical optimisation which considers the dike and dry-proofing of buildings. The set of available decision alternatives is  $A_{s,HO1} = \{a_s^{(k_1,0,k_3)}, k_1 = 1, 2, 3, k_3 = 1, \dots, N_3\}$ .
- Decision optimisation HO2 – Two-level hierarchical optimisation which considers the dike and local flood barriers. The set of available decision alternatives is  $A_{s,HO2} = \{a_s^{(k_1,k_2,0)}, k_1 = 1, 2, 3, k_2 = 1, \dots, N_2\}$ .
- Full hierarchical optimisation FHO – Three-level hierarchical optimisation which considers the dike, local flood barriers as well as dry-proofing of buildings. The set of available decision alternatives is  $A_{s,FHO} = \{a_s^{(k_1,k_2,k_3)}, k_1 = 1, 2, 3; k_2 = 1, \dots, N_2, k_3 = 1, \dots, N_3\}$ .

In each analysis, optimal configuration of the hierarchical flood protection system is identified using the available decision alternatives.

Results of the optimisation are presented as follows; in Figure 7.16, the optimal system configurations for each decision analysis are mapped out; in Figure 7.17, exceedance probability curves are compared; Figure 7.18 compares the optimal system configurations for each decision analysis in terms of annual expected loss, construction cost and expected utility. Finally, Table 7.5 summarises key characteristics of each optimal system configuration.

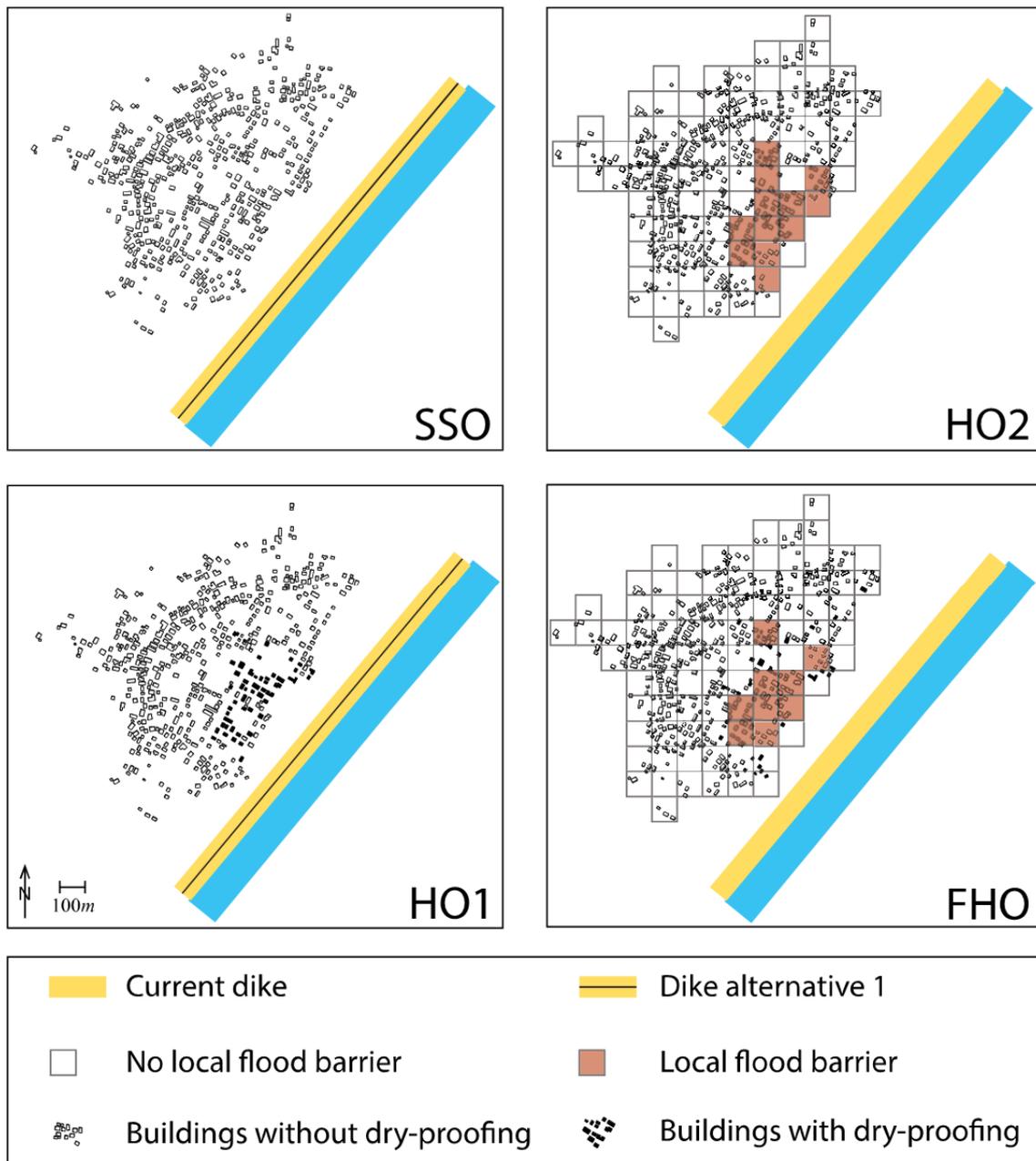


Figure 7.16 - Optimal configurations of the hierarchical flood protection system for different decision constraints

As illustrated in Figure 7.16, the decision alternative is part of the optimal system configuration and seems to depend on whether local flood barriers are available in the decision optimisation: when they are not available, dike decision alternative  $a_1^{(1)}$  is part of the optimal system configuration. Conversely, when local flood barriers are available, the dike alternative  $a_1^{(0)}$  seems to suffice. Dike alternative  $a_s^{(2)}$  is never part of an optimal system configuration.

The decision optimisations that included local flood barriers and/or dry-proofing seem to indicate that these protection measures are particularly efficient in proximity of

the river, where the probability of flooding is largest. However, the buildings in the South-Western corner of Kriessern are close to the river and yet do not seem to require protection from local flood barriers and dry-proofing. The reason might be found in the terrain, which is slightly more elevated in that area.

The full hierarchical optimisation (FHO) yields a system configuration where no one building is protected by all three protection levels. This observation is further indication that the marginal increase in expected utility from the addition of a protection level decreases with increasing number of protection levels.

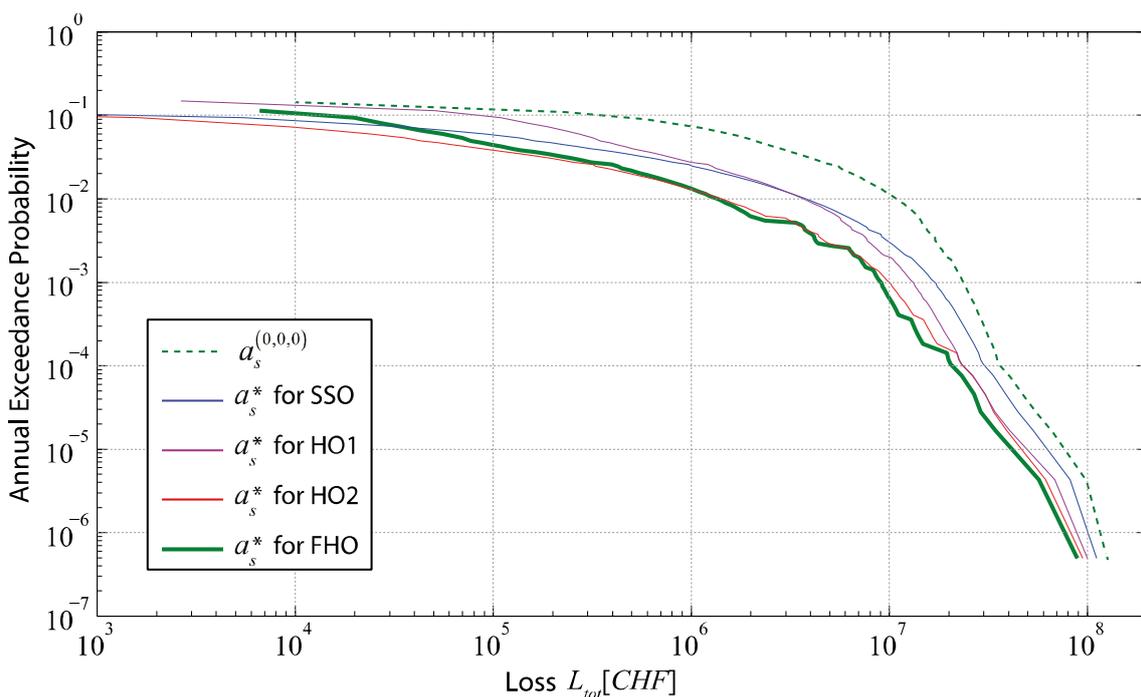


Figure 7.17 - Annual exceedance probability of loss  $L_{tot}$  for the current state of the protection system  $a_s^{(0,0,0)}$  and the optimal system configuration  $a_s^*$  for each decision analysis

The EP-curves comparison indicates that, compared to the present protection system, the optimal system configuration from each decision analysis reduces losses at all exceedance probabilities. However, each optimal system configuration reduces risk differently. For instance,  $a_{s,SSO}^*$  ( $a_s^*$  for SSO) is more effective than  $a_{s,HO1}^*$  for small events with return periods below ca. 100 years, but less effective for larger events. Similar observations are made comparing  $a_{s,HO2}^*$  and  $a_{s,FHO}^*$ . As such, the comparison of EP-curves does not allow an easy identification of the system that entails least risk. Here, Figure 7.18 is of help, it summarises total risk reduction, total construction costs and expected utility of each optimal system configuration. Table 7.5 gives further details on the characteristics of each optimal system and of risk reduction and costs they

entail. As expected, the expected utility of the optimal system configuration increases with the number of considered protection levels. Full hierarchical optimisation leads to the largest expected utility; single-structure optimisation to the smallest.

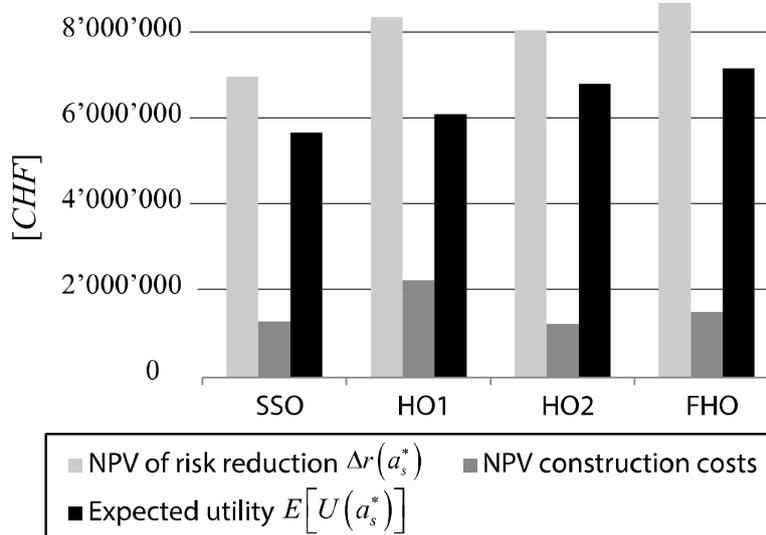


Figure 7.18 - Net present value (NPV) of risk reduction and construction costs as well as the expected utility for the optimal system configuration from each decision analysis

Table 7.5 - Characterisation of the current protection system configuration and the optimal system configuration for each decision optimisation

System configuration	Current	SSO	HO1	HO2	FHO
Dike decision alternative	$a_1^{(0)}$	$a_1^{(1)}$	$a_1^{(1)}$	$a_1^{(0)}$	$a_1^{(0)}$
Number of local flood barrier	0	0	0	12	9
Number of dry proofed building (percentage of total buildings)	0	0	50 (10.4%)	0	28(5.8%)
<b>Risk</b>					
NPV $r_{tot}$ [CHF / year ]	10'214'000	3'196'800	1'814'540	2'113'500	1'475'680
NPV $\Delta r$ [CHF / year ]	0	7'017'200	8'399'460	8'100'500	8'738'320
<b>Construction costs</b>					
NPV dry-proofing costs [CHF]	0	0	954'660	0	528'308
NPV local flood barrier costs [CHF]	0	0	0	1'245'658	996'526
NPV dike costs [CHF]	0	1'302'000	1'302'000	0	0
NPV total construction costs [CHF]	0	1'302'000	2'256'660	1'245'658	1'524'834
Expected utility [CHF]	0	5'715'200	6'142'800	6'854'842	7'213'486

## 7.6 Discussion

The considered set of decision alternative  $A_s$  offers decision makers a range of system configurations with markedly different risk reduction potential. Based on Figure 7.15, comparison of EP-curves of the decision alternative entailing the largest risk ( $a_s^{(0,0,0)}$ ) and the decision alternative entailing the smallest risk ( $a_s^{(2,N_2,N_3)}$ ) suggest that flood risk for the latter is approximately 100 times smaller than for the former. However, this comparison does not consider construction costs; in all probability, system configurations entailing smaller risk would entail very large construction costs, making them economically unattractive.

In Figure 7.15, EP-curves for system configurations with local flood barriers show significant discontinuities ("jumps") in the loss level. The reason for these is explained in the following at the example of individual local flood barrier behaviour. Flood events where water level does not reach the top of the barrier cause no damage in the area protected by a local flood barrier. Conversely, when a flood event overtops a particular barrier, the protected area is assumed fill up instantaneously to the water level it would have had, without a local flood barrier in place, see Equation (7.5). A discontinuity in the EP-curve appears between two events, when the water level of the first event does not overtop the local flood barrier and the water level of the second event overtops the local flood barrier.

Results illustrate the potential of hierarchical flood protection systems in flood risk management, as such systems allow for flexibly tailoring flood protection in function of hazard and receptors.

Optimisation of hierarchical flood protection system allows identifying protection system configurations with a larger expected utility than single-structure optimisation. Results also indicate that the effectiveness of a flood protection system in reducing risk increases with a greater number of protection levels. That is, the expected utility of  $a_{s,SSO}^*$  is smaller than the expected utilities of  $a_{s,HO1}^*$  and  $a_{s,HO2}^*$ , which are in turn smaller than the expected utility of  $a_{s,FHO}^*$ . The reason for the increase in expected utility with increasing number of protection levels is that every additional protection levels allows for better tailored flood protection.

Local flood barriers and a strengthened dike  $a_1^{(1)}$  seem to reduce risk in similar manner. In decision analyses which did not consider local flood barriers (SSO and HO1), optimal system configurations include the strengthened dike  $a_1^{(1)}$ , which allowed for reduction of the probability of flooding. Conversely, in decision analyses including

local flood barriers (HO2 and FHO), the current dike  $a_1^{(0)}$  seems to be sufficient, as local flood barriers can be installed to reduce probability of flooding where necessary.

The example application also allowed for testing of the vulnerability model presented in Chapter 6 in a decision analysis context. The vulnerability model proved valuable, as it allowed for detailed modelling of individual buildings as well as capturing the impact of dry-proofing measure on vulnerability.

### 7.6.1 Limitations

This analysis aimed at testing and illustrating the functioning of the proposed methodology for flood risk analysis and decision optimisation in the presence of hierarchical flood protection system. The analysis is limited in several regards, as data on the study area is scarce and several methodological simplifications were made to reduce the computational burden of the risk analysis.

Detailed data on river morphology and cross-sections, current protection system and receptors is not available. In addition, no, detailed hydrological data is available, necessitating the postulation of a synthetic hydrograph. In consequence, the present study can give only an approximate characterisation of flood risk in the study area and practical relevance of the results is limited.

Methodological limitations are identified in the hazard assessment, fragility models and of consequence assessment. The stationarity of the hazard source process is not assessed and the probability of having more than one event per year is disregarded. Statistical uncertainty in  $f_q(q)$  is not considered. The inflow hydrograph of each event is freely postulated with the same hydrograph shape for all events. In an improved analysis, the shape of the hydrograph should be derived from past event hydrographs (see e.g. Apel *et al.* 2006).

Flow routing models should be calibrated through comparison with data from past events along the Alpine Rhine. The impact of local flood barriers on flood development is not considered in the flow routing model. Considering different local flood barriers configurations in the flow routing number would have been unfeasible as the number of model runs would have increased significantly. However, local flood barriers of this size are likely to impact flood development in nearby areas, and should thus be considered in the flow routing model. Furthermore, the overtopping process of local flood barriers should be considered in more detail, allowing for a more gradual flooding of areas protected by local flood barriers.

Modelling of breaching scenarios can be improved by considering dike breach geometry probabilistically. Furthermore, scenarios entailing two or more breaches should be considered (see e.g. Dawson *et al.* 2005). Dike fragility models incorporating dike characteristics should be implemented to allow for detailed consideration of failure mechanisms. Furthermore, the assertion that local flood barriers do not fail, should be further investigated and, if necessary, a fragility model should be developed to characterise breaches in local flood barriers.

As stated in Chapter 2, for consistent decision making, it is important to consider all types of consequences. Here, consequence assessment is limited to direct tangible consequences of one asset category, residential buildings. By disregarding other consequences, results may be misleading and favour protection system configurations relying on focused protection of valuable assets, rather than large protection structures providing extensive protection for the whole flood plain. If the same analysis is repeated with consideration for further consequences, e.g. the damage to infrastructure and to the agricultural sector, large protection structures would probably be favoured over small and focused protection structures.

In the decision analysis, three alternative dike configurations are considered. In practice, relevant resistance parameters and protection height are continuous variables, which can be optimised with much more precision. Similarly, decision alternatives considered on the second hierarchy level are artificially constrained, as only one type of local flood barrier is considered and placement of barriers on grid cell borders is postulated.

Decision optimisation is based only on economic optimality of decision alternatives. In an improved analysis, further aspects should be considered, for instance, robustness of the flood protection system configuration in regard to dike breaches, minimisation of the failure probability and flexibility for adaption for changing future flood risk.

Several methodological simplifications (in particular, hazard is modelled deterministically and protection structure breaching is considered only on one hierarchy level) allowed circumventing challenges inherent to flood risk analysis in the presence of hierarchical flood protection systems. In particular, they enabled reduction of the flow routing model runs. For the application of the proposed methodologies in practice, such simplifications are not permissible. As such, further research is necessary to allow for the implementation the methodology proposed in Chapter 4 without these simplifications.

## **8 Conclusions and outlook**

This chapter concludes the thesis by summarizing and discussing its key contributions, as well as giving recommendations for future research.

### ***8.1 Originality, limitations and recommendations for future research***

The original contributions of this thesis are found in Chapters 4 through 7. As the chapters primarily discuss distinct topics, original contribution and limitations - as well as suggestions for future research - are summarised for each chapter separately in the following, and a more general conclusion and research outlook is provided thereafter.

Chapter 4 outlines a modelling approach for flood risk assessment in the presence of a hierarchical flood protection system. The original contributions of Chapter 4 are:

- A definition and characterisation of hierarchical flood protection system is provided.
- The advantages of hierarchical flood protection system are described. They include: residual risk management, robustness, tailoring and the planning of future risk reduction capacity. At the same time potential disadvantages are highlighted, in particular flawless planning and execution are paramount for hierarchical flood protection system to deliver the promised advantages.
- A methodology is proposed for flood risk assessment in the presence of a hierarchical flood protection system, as well as for its optimisation.
- Several challenges encountered when modelling hierarchical flood protection systems are described.

The main limitations of the work presented in Chapter 4 are:

- The proposed methodology for flood risk analysis in the presence of hierarchical flood protection system hinges on the assumption of conditional independence between several variables. In certain situations, this assumption might not hold.
- The modelling of hazards in accordance with the proposed methodology might be challenging, particularly when a distributed flow routing model is utilised.
- Two advantages of hierarchical flood protection system are robustness and flexibility. However, the proposed utility function does not capture either, focusing only on optimal risk reduction.

Further research recommended in regard to the work presented in Chapter 4 includes:

- Find a solution to the outlined implementation challenges.
- Establish and test utility functions, which consider not only economic cost-benefit, but also robustness and flexibility of the flood protection system.
- Analyse how hierarchical flood protection systems perform in regard to uncertainty of future flood risk and whether the consideration of hierarchical flood protection systems allows developing flood risk management strategies which can be flexibly adapted to the requirements of a changing flood risk.

- Identify practical situations (geography, river course, catchment characteristics, etc.) where a hierarchical flood protection system might be particularly useful.

Chapter 5 introduces a model for the disaggregation of spatially aggregated amounts, which can be utilised to probabilistically disaggregate an aggregated portfolio of receptors. The original contributions in Chapter 5 are:

- The probabilistic disaggregation model is a contribution towards consistent natural hazard risk modelling at different spatial scales and resolutions.
- The model allows disaggregating an aggregated amount in accordance with an indicator available at a spatially higher resolution under consideration of uncertainty.
- The spatial correlation between disaggregated variables is considered through a Gaussian copula. As such, the model allows for separately defining marginal probability distribution of disaggregated variables and their correlation structure.
- The impact of disaggregation uncertainty on natural hazard risk assessment is illustrated through the example of a flood risk assessment. When disaggregation uncertainty is duly considered, the tails of the risk distribution increase as expected.

The limitations of the presented probabilistic disaggregation model include:

- The size of a disaggregation problem is limited by computation of the correlation matrix of the multivariate normal distribution.
- The disaggregation model is only tested for disaggregation to raster grid cells. Whereas this is often sufficient for natural hazard risk analysis, the disaggregation to cells with different geometry should be tested.
- Correlation between grid cells with different indicator values is not reproduced correctly.
- Whereas the behaviour of the model is characterised with a numerical study, an analytical closed form of the joint probability function of disaggregated variables is not available nor is the analytical formulation of the variance of disaggregated variable.

Recommended further research includes:

- The probabilistic disaggregation model should be tested for applications with non-rasterised disaggregation areas.
- Further investigation of the joint probabilistic characterisation of disaggregated variables, including research towards the identification of an analytical closed form of the joint probability density function is recommended.

- The impact of probabilistic disaggregation model on natural hazard risk assessment should be analysed, e.g. by applying the disaggregation model to realistic risk assessments.

Chapter 6 introduces a modelling approach for the flood vulnerability assessment of residential buildings by explicit damage process modelling. The main contributions of the modelling approach are the following:

- The modelling approach identifies relevant damage process (i.e. water infiltration into the building, mechanical failure of components in the building envelope and damage from water contact) and represents them in probabilistic manner to obtain a vulnerability curve.
- The modelling approach is abstracted and modularised, which makes it in principle applicable to a large variety of building types and hazard situations.
- Building characteristics are considered in extensive detail to capture the change in vulnerability introduced by flood proofing measures; the vulnerability model can be used as a basis for decision analysis.
- The modularity of the model allows for improving the vulnerability assessment when new knowledge or new modelling approaches become available, e.g. in regard to individual damage processes.
- The probabilistic approach and consistent treatment of uncertainty allows for modelling vulnerability consistently with the available information on the building configuration. As such, the modelling approach allows for coherently modelling vulnerability curves for single buildings, as well as for portfolios of buildings.
- The proposed modelling approach is implemented for 1- and 2-storey masonry buildings; it seems to perform according to expectations.

Limitations of the vulnerability modelling approach and its implementation are:

- The modelling approach neglects several failure modes, such as buoyancy and foundation score and does not consider all hazard characteristics, like flood duration, debris flow and water contamination. However, the modular structure of the model allows these to be added in the future.
- The modelling approach generally assumes that buildings are characterised in great detail, which necessitates the estimation of a large number of parameters and requires significant effort for establishing, implementing and validating the model.
- Although it is claimed that the modelling approach is applicable to a variety building types, it is not implemented for reinforced concrete and timber buildings; its validity for these building types cannot be confirmed.

- In the implementation of the modelling approach, damage processes are modelled with simplified modelling approaches.
- Full validation of the modelling approach and its implementation, beyond the anecdotal attempt made in Chapter 6, is challenging, since it would require an analysis of results for a multitude of possible parameter combinations.

The following points are recommended for future research:

- The applicability of the modelling approach for other building types, e.g. concrete or timber buildings should be demonstrated.
- A sensitivity analysis on the number and type of considered components should be undertaken.
- Damage mechanisms not considered here should be included in the modelling approach.
- Further validation of the modelling approach, as well as its implementations, is required before the approach can be applied in practice.
- The models of individual damage processes should be refined and independently validated.
- A rationale for the estimation of the large numbers of parameters should be established.
- The consideration of the hazard indices should be improved, i.e. the modelling approach should be improved to consider real hazard event time series as a hazard input.

In Chapter 7, the hierarchical flood protection system is implemented for the village of Kriessern, Switzerland; main contributions of the chapter are:

- The concepts of hierarchical flood protection system, the methodology for flood risk assessment in the presence of a hierarchical flood protection system and the vulnerability model for residential buildings in floods are tested.
- As anticipated, results indicate that hierarchical flood protection systems have larger expected utility than single-structure flood protection systems. The results further suggest that the efficiency of a flood protection system in reducing risk increases with increasing number of hierarchy levels.
- The vulnerability model specified in Chapter 6 seems to perform as expected as part of a decision analysis, allowing evaluation of flood proofing measures efficiency for individual building and hazard situation

The limitations of the example application are:

- The practical relevance of the described risk assessment is questionable, since the quality of available data is poor and several assumptions had to be made.

Data on receptors, river and current protection system are not available in sufficient detail for an accurate risk assessment.

- Several methodological aspects were simplified to reduce computational times, e.g. the geometry of dike breaches; the shapes of synthetic hydrographs are considered deterministically. Furthermore, breaching scenarios with more than one breach are neglected.
- Consequence assessment considers only direct tangible consequences for residential buildings. Indirect and intangible consequences are neglected, as are consequences to other asset categories. In all likelihood, a more complete consequence assessment would shift the preference towards a higher and stronger dike rather than local flood barriers and dry-proofed buildings.

Further research in regard to the example application may include:

- Given the identified limitations, the flood risk assessment and decision analysis should be carried out based on better data to confirm the findings, and ultimately, the value proposition of hierarchical flood protection systems.

### **8.2 Outlook**

Here, a more general outlook is given on further research necessary to utilise the concept of hierarchical flood protection systems in practice.

The disaggregation model presented in Chapter 5 and the vulnerability model in Chapter 6 both contribute to the consistent modelling of flood risk at different spatial resolutions, however, challenges remain to achieve this goal. In particular, for large study areas, risk assessment for optimisation of a hierarchical flood protection system might require hazard to be assessed at different resolutions, with different flood routing models. Research may be required to assure that hazard modelling at different resolutions is consistent.

When breaching scenarios are considered on more than one hierarchy level, the number of unique breaching scenarios grows exponentially with the number of hierarchy levels, posing a computational challenge, which is not encountered in the example application, since breaches are only considered on one hierarchy level. Further research is necessary to identify computational methods to model a large number of hazard scenarios under due consideration of breaches in protection structures on more than one hierarchy level.

The concept of hierarchical flood protection system promises flexibility in adapting a flood protection system to changing flood risk. In this context, research work should

be undertaken to analyse potential advantages of hierarchical flood protection systems in regard to climatic uncertainty.

In this thesis, one application of hierarchical flood protection system optimisation is carried out, with the example in Chapter 7. The utilisation of hierarchical flood protection systems in other hazard contexts, e.g. mountain river and sea flooding, different protection structures, including a dam, and for larger study areas, should be tested. In Chapter 7 the optimisation of hierarchical flood protection system is approached with the objective of optimising expected utility, i.e. the economic benefit of the risk management measure. In future research, optimisation of hierarchical flood protection systems with different utility functions should be analysed, i.e. optimising for adaptability for future changes in flood risk or robustness.

### **8.3 Concluding remarks**

Flood risk management is a continuous process of risk assessment, risk treatment and risk monitoring, with periodical re-evaluation and adaption to account for changing flood risk and improved assessment of methodologies. This thesis aims at improving flood risk management by proposing and formalising the concept of hierarchical flood protection system and by making two methodological contributions to improve flood risk assessment.

Taking integrated flood risk management as a starting point, hierarchical flood protection systems are characterised and formalised, and a methodology for flood risk assessment in presence of hierarchical flood protection system is proposed. Whereas such flood protection systems are common in practice, their advantages are seldom formally considered in flood risk management. By characterising hierarchical flood protection systems and detailing their advantages, the potential of hierarchical flood protection systems is highlighted for both researchers and practitioners. Their mathematical formalisation and the flood risk assessment methodology provide tools for further research and for considering such protection systems in practice.

Furthermore, this thesis proposes two new methodological approaches to particular aspects of flood risk assessment: consideration of uncertainty in disaggregation of spatially aggregated portfolio of receptors and modelling of residential building vulnerability. The probabilistic disaggregation model allows for disaggregating spatially aggregated portfolios of exposure with due consideration of disaggregation uncertainty.

The consideration of this type of uncertainty in natural hazard risk assessment is relevant, allowing for modelling risk distributions tails more accurately.

The proposed modelling approach for vulnerability of residential buildings is general in its application and can represent the impact of flood proofing measures on building vulnerability. It can thus be utilised in a formal engineering decision analysis. The modelling approach provides a basis for researchers to develop consistent damage process-based vulnerability models for different building types. In practice, vulnerability models based on the proposed modelling approach can be utilised to determine the effectiveness of flood proofing measures on residential buildings, thus allowing for a formal decision analysis.

Hierarchical flood protection systems can make an important contribution to improved flood risk management and allow a sensible use of societal resources. This thesis provides several building stones to facilitate the modelling of hierarchical flood protection systems in research and practice.

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In this thesis hierarchical flood protection systems (multiple lines of defence) are investigated. Hierarchical flood protection systems offer several advantages compared to single-structure flood protection systems, since they can be precision-tailored to fit risk reduction requirements and allow for flexible adaption of the protection system to changing flood risk. As part of the thesis, a new methodology to model flood vulnerability of residential buildings by explicit damage process is presented and a methodology to disaggregate a spatially aggregated portfolio of buildings.

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