Title:	**Effects of River System Behaviour on Flood Risk**

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Abstract

The aim of this study was the development of a Conceptual Framework for quantifying effects of river system behaviour on flood risks in order to allow *policy makers* to evaluate the consequences that particular safety-improvement measures have on the overall safety within a catchment. The developed Conceptual Framework is based on an integral flood risk analysis; the overall product of probability of flooding and its consequences. It consists of an Institutional Framework and a Computational Framework. Although several institutional aspects are briefly outlined, the study mainly focussed on developing the Computational Framework.

With effects of river behaviour is meant that the safety of a particular dike-ring might depend on the safety of other dike-rings. These effects might be positive or negative. Positive effects of system behaviour might occur in case the hydraulic load on a dike-ring is reduced due to the failure of an upstream located dike-ring. Negative effects of system behaviour might occur in case due to a local dike breach, flood water from a major river branch flows into a minor river branch (f.i. from Waal to Meuse). The driving mechanism in system behaviour is the mutual interaction between geo-technical failures of dikes and the hydrodynamic response of the river system to it.

The Computational Framework includes hydraulic modelling of river system behaviour, modelling of geo-technical failure mechanisms, flood risk analysis and flood-caused damage assessment. Using a simplified version of the Computational Framework two hypothetical but realistic case studies were conducted. Despite the simplifications it can be stated that, a <u>significant</u> step towards a more sophisticated Computational Framework was made. The first case study demonstrated that calculated flood risk reduces (i.e. positive effect) when system behaviour is taken into account. A case study focussing on demonstrating negative effects of system behaviour was not conducted. The second case study demonstrated that a deterministic design approach for an emergency retention polder that does not take uncertainties into account, may yield incorrect indications of the flood risk, and may, therefore, be misleading in the evaluation of flood protection measures. Statistical analysis showed that the applied Monte Carlo analysis provides sufficient accuracy in computed flood risk. It was concluded that for determining <u>proper</u> flood risks, it is a *prerequisite* that effects of system behaviour, geo-technical failure mechanisms, uncertainties and safety-improvement are jointly assessed. The developed Computational Framework provides this required functionality.

Current Dutch practice is to assess flood-safety on basis of single dike-sections or a complete dike-ring. Mutually interactions between various dike-rings are not yet considered. Taking the above into account, it is strongly recommended to apply in future flood-risk assessments the concept of the Computational Framework.

It is to be mentioned that the Computational Framework applied for conducting both case studies still is subject to further improvement. Several recommendations are given in this respect.

PROJECT NAME:	**Effects of River System Behaviour on Flood Risk**	PROJECT CODE:	**02.01.01**
BASEPROJECT NAME:	**Loads on water-retaining structures**	BASEPROJECT CODE:	**02.01**
THEME NAME:	**Risk due to flooding**	THEME CODE:	**01**

Executive Summary

Brief description of project activities:
In this study a concept of flood risk evaluation has been applied to quantify effects of river system behaviour in a regional setting. The concept (or Conceptual Framework) is based on an integral catchment-wide flood risk analysis; the overall product of probability of flooding and their consequences. The Conceptual Framework consists of an Institutional Framework and a Computational Framework. Although several institutional aspects have been briefly outlined, the study has mainly focussed on the development of the Conceptual Framework.

With effects of river behaviour is meant that the safety of a particular dike-ring might depend on the safety of other dike-rings. These effects might be positive or negative. Positive effects of system behaviour occur in case the hydraulic load on a dike-ring is reduced due to the failure of an upstream located dike-ring. Negative effects of system behaviour occur in case due to a local dike breach, flood water from a major river branch flows into a minor river branch (f.i. from Waal to Meuse). The driving mechanism in system behaviour is, of course, the mutual interaction between geo-technical failures of dikes and the hydrodynamic response of the river system to it.

Using a simplified version of the Computational Framework two hypothetical but realistic case studies were conducted. The first case study referred to a system without any human interference. In the second case study the operation of an emergency retention polder aiming at mitigating the downstream flood risk was taken into account. This Computational Framework consisted of: hydraulic modelling of river system behaviour (i.e. one-dimensional river flow and two-dimensional polder (or dike-ring) flow; modelling of geo-technical failure mechanisms (i.e. overtopping and piping); flood risk analysis (i.e. Monte Carlo analysis) and flood-caused damage assessment (i.e. Dutch Standard Damage model, HIS-SSM, considering economic damage only).

Conclusions:
The study resulted in a good insight into the requirements of the Computational Framework. Requirements that concern the incorporation of hydraulic aspects, geotechnical and structural aspects, flood risk aspects, and uncertainties in physical processes as well as in societal and institutional aspects.

Due to practical limitations and the current state of required technology/knowledge, simplifications in the Computational Framework applied for conducting the two case studies were made. Nevertheless, a significant step towards a more sophisticated Computational Framework was made.

Case study no 1 demonstrated that calculated flood risk reduces (i.e. positive effect) when system behaviour is taken into account. Further on that for single and flood-wave dominated rivers having same protection standards yields that downstream located polders benefit more from effects of system behaviour than upstream located polders. Please note that a case study focussing on demonstrating negative effects of system behaviour was not conducted.

Case study no 2 demonstrated that a deterministic approach for designing an emergency retention polder, meaning that uncertainties in geo-technical failure mechanism, magnitude of upstream discharges and so on are not taken into account, may yield incorrect indications of the flood risk, and may, therefore, be misleading in the evaluation of measures to mitigate flood risk.

More general conclusions from conducting both case studies are:

1. The Conceptual Framework was generic enough for conducting both case studies. Meaning that the uncertainties to be considered in geo-technical failure mechanisms and boundary conditions could easily be implemented. The same yielded for the operation rules considered for the emergency retention polder,
2. For properly assessing the flood risk in a particular catchment a sufficient number of flood scenario's (or Monte Carlo runs) is to be made,
3. That part of the catchment area is to be considered, that is prone to effects of system behaviour and surrounding the area where a particular safety-improvement measure will be implemented. This is necessity for properly assessing the consequences of such safety-improvement measure on resulting flood risks. In addition the boundary conditions of the system should be independent for effects of system behaviour. If not these effects have to be accounted for in the applied boundary conditions,
4. For properly assessing the flood risk in a particular catchment, it is essential to consider all relevant failure mechanisms as well as all proposed safety-improvement measures jointly.

From statistical analysis on the case study (more precisely: Case study no 1, option 1) results, it was shown that the applied Monte Carlo analysis provides sufficient accuracy in computed flood risk. A characteristic feature of flood damage in our calculation appeared to be that it depends significantly on the circumstances and conditions which lead to dike breach. That is, in the calculation the flood damage may be seriously affected by the actual values of the stochastic parameters. This is particularly true for flood damage originating from simultaneous dike breaches at different locations (in the same or in different areas) where inflow of river water through the breaches is affected by the river system behaviour. The adopted Monte Carlo approach takes this into account.

Concluding:

For determining <u>proper</u> flood risks, it is a *prerequisite* that effects of system behaviour (both passive and active interference by mankind), geo-technical failure mechanisms, uncertainties and safety-improvement are jointly assessed. The Computational Framework provides this required functionality.

Recommendations:

Current Dutch practice is to assess flood-safety on basis of single dike-sections or a complete dike-ring. Mutually interactions between various dike-rings are not yet considered. Taking the conclusions given above into account, it is strongly recommended to determine in future flood-risk in line with the concept of the explained Computational Framework, that allows for taking into account effects of system behaviour on flood risk. This will introduce the need of a model combining the hydraulic modelling of river system behaviour, geo-technical failure mechanisms, uncertainties in various model parameters, as well as operational options of various structures accommodated in the river system. Using such model the consequences of a particular safety-improvement for the entire catchment area can be assessed. Such model should be maintained and constantly be updated for newly implemented safety-improvement measures.

It is to be mentioned that the Computational Framework applied for conducting both case studies still is subject to further improvement. In this report several recommendations for further improvement are discussed. In short these recommendations are:

<u>*Related to hydraulic modelling*</u>:

1. To improve the computational efficiency by introducing parallel processing and domain decomposition into SOBEK-Rural/Urban,

2. To develop a post-and pre-processing tool for reducing the effort in making and analysing the several hydraulic computations,
3. To model the river as two dimensional flow in order to leave to the system to find the most appropriate location for dike-failure or overtopping,
4. To develop an algorithm capable of determining beforehand for complex river systems such as the Dutch river delta for which scenario's no flooding is to be anticipated. Hence making it possible to reduce the number of scenario's to be computed,

   Related to Geotechnical and structural aspects:

5. To include additional geo-technical failure mechanisms, such as for instance dike failure due to inner slope failure,
6. To include failure mechanisms for water-retaining hard structures,
7. To incorporate advanced formulations for geo-technical failure mechanisms, that account for the residual strength of dikes after an initial breach has occurred,
8. To incorporate improved descriptions for the development of a breach in a dike as function of actual occurring hydraulic loads,

   Related to determining flooding probability:

9. In the case studies considered the failure probabilities are so high that a straightforward (Crude) Monte Carlo analysis was performed. However, when in future more complex cases will be investigated, it might be worthwhile to adopt Importance Sampling Monte Carlo analysis in order to reduce the number of required Monte Carlo simulations,
10. In order to attain at reliable flood risk for a reduced number of Monte Carlo simulations, it is worthwhile to check for outliers (i.e. extreme sampled values) in stochastic input variables,
11. To include a preposterior Bayesian Analysis for evaluating the Monte Carlo analysis applied in determining flood risk, and

    Related to determining flood-caused damage:

12. To include water-quality aspects (spread of contaminated silt and toxic agents), indirect economic benefits, damage to environmental and cultural values, and societal aspects in the evaluation of flood risks.

Both in policy making and during crisis situations the concept of system behaviour will have influence on decision making. In this study a very limited analysis of institutional aspects has been carried out. A further investigation of these aspects is, therefore, recommended.

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General Appendix: Delft Cluster Research Programme Information

This publication is a result of the Delft Cluster research-program 1999-2002 (ICES-KIS-II), that consists of 7 research themes:
1) Soil and structures, 2) Risks due to flooding, 3) Coast and river , 4) Urban infrastructure, 5) Subsurface management, 6) Integrated water resources management, 7) Knowledge management.

This publication is part of:

Research Theme	:	Risk due to flooding		
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Delft Cluster is an open knowledge network of five Delft-based institutes for long-term fundamental strategic research focussed on the sustainable development of densely populated delta areas.

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Name	Organisation
Not relevant	Not relevant

Effects of River System Behaviour on Flood Risk

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E **Detailed Results of Case study no 2**

F **Preposterior Bayesian Analysis**

Summary

The aim of this study was the development of a Conceptual Framework for quantifying effects of river system behaviour on flood risks in order to allow *policy makers* to evaluate the consequences that particular safety-improvement measures have on the overall safety within a catchment. The developed Conceptual Framework is based on an integral flood risk analysis; the overall product of probability of flooding and its consequences. It consists of an Institutional Framework and a Computational Framework. Although several institutional aspects are briefly outlined, the study mainly focussed on developing the Computational Framework.

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Current Dutch practice is to assess flood-safety on basis of single dike-sections or a complete dike-ring. Mutually interactions between various dike-rings are not yet considered. Taking the above into account, it is strongly recommended to apply in future flood-risk assessments the concept of the Computational Framework.

It is to be mentioned that the Computational Framework applied for conducting both case studies still is subject to further improvement. Several recommendations are given in this respect.

1 Introduction

1.1 Background

The Delft Cluster Project DC 02.01.01: Effects of System Behaviour on Flood Risk was conducted by the following five Delft Cluster (DC) partners, viz:

1. WL | Delft Hydraulics (Project leader Ir. M.C.L.M. van Mierlo, Ir. K.M. de Bruijn and Dr. Ir. A.H. Weerts),
2. DWW (Dienst Weg- en Waterbouwkunde) of the Dutch Ministry of Public Works (Ir. S.N. Jonkman).
3. GeoDelft (Ir. E.O.F. Calle),
4. TNO Bouw (Prof. Ir. A.C.W.M. Vrouwenvelder), and
5. TU Delft, Department of Civil Engineering and Geo Sciences (Prof. Ir. J.K. Vrijling)

1.2 Aim

The aim of the DC 02.01.01 project was the development of a methodology or Conceptual Framework for quantifying the Effects of System Behaviour on Flood Risk. With effects of system behaviour on flood risk is referred to the fact that the flood risk (or safety) of a particular area can depend on the safety of other areas. It is possible that a measure for improving the safety of a particular area might increase or decrease the safety of other areas, located within the same hydrological system (or catchment area).
The purpose of the Conceptual Framework is to support *policy-makers* in evaluating the consequences that particular safety-improvement measures have on the overall safety within a catchment. Hence, the use of the Conceptual Framework, will allow policy-makers to apply an integral safety-approach. Meaning that for an improvement measure not only its safety-consequences for a particular area is considered, but that its safety-consequences for all areas within the catchment are considered and that on basis thereof a decision can be made whether such measure should be implemented.
An additional condition with respect to the Conceptual Framework was that it should be generic. Meaning that it can be applied to any hydrological system, among others the complex Rhine and Meuse river system in the Dutch delta. In addition it should be possible to consider any kind of safety-improvement measure, such as: increase of conveyance capacity ("room" for the river), temporarily storage of floodwater in retention areas (noodoverloopgebieden), heightening of existing embankments and so on.

1.3 Approach and scope

The following approach was adopted in developing the Conceptual Framework. First an inventory was made of all (technical and non-technical) aspects, that are of importance for quantifying the effects of system behaviour on flood risk. Thereafter, an inventory was made of existing methods/tools for assessing the effects of these aspects on flood risk. Based on the findings of these inventories the Conceptual Framework was divided into an Institutional Framework and a Computational Framework. The Institutional Framework should deal with societal aspects (f.i. acceptance by society of safety standards and related system behaviour) as well as policies and decision making aspects. The Computational Framework should allow for assessing the effects on flood risk of any kind of safety-improvement measure in any kind of hydrological system.

In the course of the project it was decided to focus first on developing the Computational Framework and evaluating its suitability by conducting two different case studies. The reason for doing so was the consensus that first a more profound understanding of system behaviour and it effects on flood risk was needed, before the Institutional Framework could efficiently be defined. Due to this approach only limited attention was paid to institutional aspects and as a result of this the Institutional Framework was not elaborated.

1.4 Readers guide

In Chapter 2 first the current Dutch practice in accounting for effects of system behaviour on flood risk is discussed. Thereafter a definition of what is meant with effects of system behaviour on flood risk is given. Finally aspects of importance in system behaviour are discussed. In Chapter 3 a Conceptual Framework, comprising of a Computational Framework and an Institutional Framework is described. This Conceptual Framework is required for determining the effects of system behaviour on flood risk. A detailed outline of the Computational Framework is given. Only limited attention was paid to institutional aspects. In Chapter 4 a description is given of two case studies that were conducted for assessing the suitability of the Computational Framework. In Chapter 5 the results and findings of these two conducted case studies are discussed. In Chapter 6 the Computational Framework applied for establishing the flood risk for the two case studies is evaluated. In Chapter 7 main conclusions and recommendations for future improvement of the Conceptual Framework are given.

2 Effects of System Behaviour on Flood Risk

In this Chapter first a description is given of the current Dutch method in determining flood risk. Thereafter, a definition of what is meant with the "Effects of System Behaviour on Flood Risk" is given. Finally an overview of aspects that are to be considered in system behaviour is given.

2.1 Current Dutch method in determining flood risk

In the current Dutch flood protection strategy, dikes are dimensioned based on a design water level with a certain probability of exceedance. The required protection levels differ by dike-ring area. Dike-rings next to the border with Germany have a design water level with a return period of 1250 yrs, while dike-rings situated further downstream have higher protection standards (1/2000, 1/4000 and 1/10.000 yrs). These protection standards are based on the economic value of the areas. The corresponding design water levels are determined on basis of the hydrodymamics of the river system and the probability density functions of sea-levels and upstream river discharges. Hence, the considered effects of system behaviour are constrained to the propagation of tidal waves and flood waves in the river delta (excluding effects of inundations) only. It is to be mentioned that in determining the design water levels for the downstream areas, upstream discharges of 18000 m^3/s at Lobith are considered. Such discharge will presently lead to large inundations in the upstream areas and hence will never reach the downstream areas. Hence in this respect system behaviour is ignored.

The current Dutch method in determining flood risk does not account for effects of system behaviour as hereafter described in section 2.2. However, in several studies different aspects of system behaviour were considered, viz:

1. Jager, F.G.J de (1998),
2. Picaso (2000-2001),
3. Tillie, J. (2001),
4. Vermeij, M (2001),
5. Hao-Ming Zhou (1995),
6. IRMA Living with floods (2001),
7. VNK/FLORIS (2003)
8. The Room/space for rivers program,
9. Spankracht study, and
10. Commission Luteijn (2001-2002).

For more information on the degree in which particular aspects of system behaviour were taken into account in these studies, reference is made to Appendix A.

The history of dealing with flood risk started with damage reduction and evolved towards reducing only the flooding probability. Presently, these two lines of flood management seem to merge as we are now in a situation were flood risk is dealt with by taking into account both the flooding probability and damage reduction (citation Parmet (2003)).

2.2 Definition of System Behaviour

With effects of system behaviour on flood risk is referred to the fact that the flood-risk (or safety) of a particular area can depend on the safety of other areas. With respect to system behaviour, a distinction can be made between:

- Positive and negative effects (or increase or decrease of safety) of system behaviour, and
- Active and passive interference by mankind in the existing system behaviour.

It is possible that a measure for improving the safety of a particular area might increase (positive effect) or decrease (negative effect) the safety of other areas, located within the same hydrological system (or catchment area). In case of a single river system, the failure of a local embankment might result in the fact that the flood hydrograph is attenuated and hence the hydraulic load on downstream located embankments is reduced, leading to an increase of the safety of the downstream located areas. However, for more complex river networks (like the Rhine and Meuse river system in the Dutch delta) yields that the failure of a local embankment might result in an increase of the hydraulic load on downstream located embankments. An example hereof is a possible failure of the left Waal-dike near Dreumel, which except for an inundation of dike-ring 41 (Land van Maas en Waal) will probably result in an overtopping (and hence failure) of the right Meuse-dike. This will then result in a extreme large lateral inflow on the Meuse river and hence on an extreme hydraulic load on the downstream located Meuse embankments. It is to be mentioned that this situation is quite serious due to the fact that design water levels along the Waal river are about two meters above the ones in the river Meuse. More or less the same yields for the failure of the right embankment along the Upper Rhine river, that will result in additional lateral inflows on the river IJssel.

Mankind can decide to interfere (active interference) or not-to-interfere (passive interference) in the existing hydrological system. Passive interference means that mankind accepts the current system behaviour within the hydrological system. Also the natural system characteristics can change over time without direct human interference. Consider for example sea level rise and increasing discharges due to climate change. Human activities that can be discerned in a passive interference strategy refer to maintaining the existing system behaviour (f.i. maintenance of dikes, revetment walls and storm surge barriers). In active interference a distinction can be made between interference in times of flood and interference in time of non-flood. An example of an active interference measure in time of flood is the diversion of flood water by so-called green rivers, aiming at reducing the hydraulic load for areas located downstream of these diversion locations and hence increasing the safety of these areas. An example of an active interference measure in time of non-floods is for instance increasing the conveyance capacity of a river or the reinforcement of revetment walls.

Both for active and passive interference measures yields that they might have positive or negative effects. Hence, the effects of proposed safety-improvement measures on flood risk are to be considered for the entire catchment area and not only for a specific area as has often happened in the past.

2.3 Aspects in system behaviour

In analysing the effects of system behaviour on flood risk following aspects are to be considered, viz:

1. Hydraulic/hydrological aspects,
2. Geotechnical and structural aspects,
3. Flood risk aspects, and
4. Societal and institutional aspects.

2.3.1 Hydraulic/Hydrological aspects in system behaviour

For assessing the effects of system behaviour on flood risk it is necessary to model floods.
From hydraulic/hydrologic point of view one might like to consider all the water-related processes in the entire catchment area, meaning: the propagation of tidal- and flood waves in the open channel system including the effect of structures, salt, wind and ice-jamming; the possible inundation of adjacent areas accounting for the presence of vertical line-objects (f.i. primary- and secondary dikes, roads and railroads) that can act as a water-barrier; the development of an initial breach in a water-barrier as function of the actual hydraulic load; and local rainfall leading to local inundations or considerable lateral inflows. However, from a practical point of view first an assessment should be made of those phenomena which are really of importance for the flood risk assessment in a particular catchment area.
For Dutch conditions, the main problem from a hydraulic point of view might be the determination of the system boundaries, for which hydraulic conditions can be applied that are independent from downstream as well as upstream effects of system behaviour.

Except for hydraulic roughness, location and strength of vertical line-objects, lay-out and bathymetry of the river system and local drainage systems, following hydraulic characteristics (van Bendegom, 1975 and Jansen, 1979) are of importance in system behaviour:

- *The type and magnitude of the hydraulic regime*. The water movement can either be governed by the tide, the upstream river discharge or both hydraulic phenomena,
- *Uniform or non-uniform storage*. With uniform storage is referred to the fact that the water level in a river rises uniformly over its entire cross-sectional profile and that the total conveyance area is not reduced. With non-uniform storage is referred to the fact that parts of the cross-sectional area are used for storage purposes only. As soon as the water level rises above a particular intake-level, these storage areas are suddenly filled-up. Dutch rivers are based on the uniform storage principle. This in contrary to parts of the Po river in Italy. The advantage of non-uniform storage is the fact that the available cross-sectional area is applied more efficiently in attenuating the design flood wave. The disadvantage of non-uniform storage compared to uniform storage is the fact that for discharges above the design flood wave, non-uniform storage results in higher flood levels than in case of uniform storage.
- *Active- and passive regulated rivers:* With active regulated rivers is referred to the temporarily storage of flood water in reservoirs (or retention basins) or the diversion of flood water through so-called green rivers by means of regulated structures.

Passive regulated rivers refer to the same as discussed above with the notation that structures are not regulated by mankind. The considered possible use of retention basins (noodoverloopgebieden) in the Netherlands refers to either an active or passive regulated river. In addition this use of retention basins refers to the application of non-uniform storage on the land-side of the embankment,

- *Location of breach in relation to embankment length*: In case a breach occurs at the downstream part of a embankment, this might not reduce the upstream hydraulic load on both sides of the river embankments and might lead to additional breaches,
- *Unforeseen exchange of flood water between different river branches*: In complex river systems it might be possible that due to a local breach, flood water may flow from one river branch to another river branch. This can be quite dangerous in case the conveyance capacity of the receiving river branch is small compared to the magnitude of flood discharges in the delivering river branch and/or design water levels in the delivering river branch are much higher than in the receiving river branch,
- *Compartmentalisation of dike-rings*: It is to be mentioned that national and regional safety-levels can not always be determined independently of each other. For instance the compartmentalisation of a particular dike-ring can benefit the safety of a particular local area, but might result in a reduction of the overall safety in concerning catchment area. Hence regional safety-improvement measures are to be viewed on a catchment scale.
- *Uncertainties in hydraulic modelling*: In determining flood risk, the uncertainty in hydraulic modelling is to be taken into account. These uncertainties can refer to improper constructed and/or calibrated models, the use of models beyond their calibrated range and so on,
- *Flow induced transport of contaminated silt and toxic agents*: During floods contaminated silt and other toxic agents may be picked-up by the flow and later on result in unwanted depositions in inundated areas, causing large environmental problems.

2.3.2 Geotechnical and structural aspects

Analysis of hydraulic response of a complex regional system of protected areas in a river delta to extreme river discharges is pursued by a coupled system of hydraulic computation models, as explained in section 2.3.1. Basically these computation models include flow in the river system itself and flood flow in inundated areas, and the mutual interaction of these flow processes. Possible causes of inundation may be both spontaneous dike breach as well as controlled inlet of water in designated areas for temporary storage. Classical potential failure mechanisms, associated with dikes, or other hydraulic structures with water retaining function, and subsoil conditions may play a significant role in the flow processes.

Classical failure mechanisms usually associated with dikes include:

1. Overflow and wave overtopping of dike crests, initiating slope failure due to saturation of the inner slope, or initiating erosion of the inner slope revetment due to run off, in case of (too) large quantities of overtopping water.

2. Geotechnical failure of the inner slopes of dikes (in the absence of overtopping), due to gradual increase of pore pressures as a result of flow through the dike (classical slope failure),
3. Seepage erosion of the inner slopes (micro instability),
4. Breach of revetment and erosion of the outer slope due to wave attack
5. Seepage erosion of sand(y) layers below the dike (piping)
6. Geotechnical failure of the outer slope. Usually this is considered a potential mechanism of minor importance. Since it may only occur after rapid descent of the river level, it can only affect the probability of dike breach in case of a second discharge wave within the time, needed for emergency repair.

Most of the mechanisms mentioned have been depicted in Figure 2.1

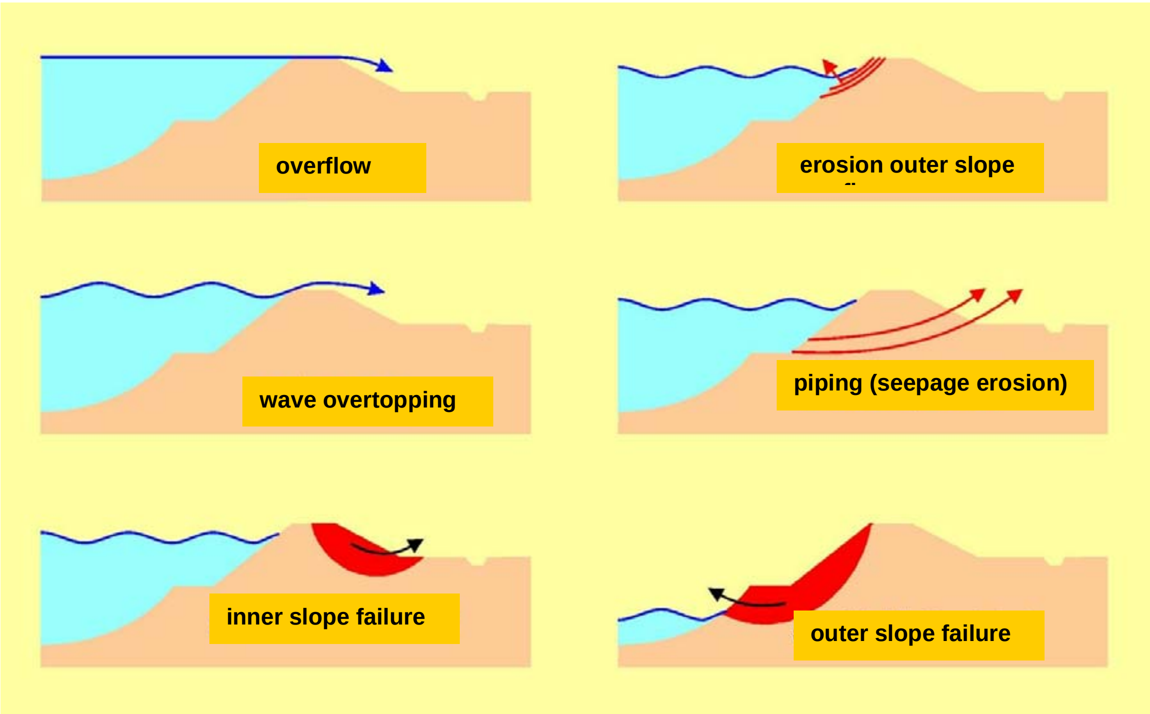

Fig 2.1 Main potential failure mechanism usually associated with dikes

Actually, the single occurrence of these mechanisms does not imply immediate dike breach or dike breach at all, since a sequence of secondary failures is needed to initiate complete reduction of water retaining capacity. For example, in the case of geotechnical failure of the inner slope, secondary, third etcetera failures are needed to finally reduce the crest height at one or more spots to a level that allows significant influx of river water. Once this stage has been reached scour will further effectuate dike breach. This is a progressive process.

The water retaining capacity of the dike after occurrence of an initial failure mechanism is usually called the residual strength. The probability of dike failure, attributable to some primary failure mechanism, equals the probability of occurrence of primary mechanism times the probability that the residuals strength is insufficient to prevent breach. Residual strength and the probability of insufficient residual strength is subject of investigation in related Delft Cluster research projects (*Failure Mechanisms and Residual Strength* and *Dike-breach Processes*). Models for progressive breach growth have been developed during earlier research, e.g. by Visser (Breach growth in sand dikes) and Verhey (breach growth in clay dikes). Summaries of the results of these two studies are given in Appendix B.

Water retaining hydraulic structures accommodated in or in between dike sections may for example consist of locks, quays, retaining walls or mobile water retaining barrages. Potential failure of the water retaining capacity of these structures is very much type specific, although generic categories may be distinguished, i.e.:

1. An unacceptable amount of water in the protected area induced by overflow or wave overtopping and, possibly, consequential structural or geotechnical failure of the water retaining structure (overtopping and overtopping induced structural failure).
2. Structural failure of components of the structure due to overload, and failure of the whole structure initiated by component failure (structural failure).
3. Loss of foundation stability due to overload, scour, piping or heave (foundation failure).
4. In case of mobile barrages, failure to establish water retaining capacity (in time), e.g. due to malfunctioning of alert protocols, inadequate handling of machinery to mobilize barrages, malfunctioning of the machinery itself or loss of power supply (failure of mobilisation system).

Furthermore, in case of systems with designated areas for temporary storage, provisions must be made for inlet of water. Various different types of inlet structures may be thought of, although not each equally suitable. The effectiveness of flood control through temporary storage depends on a variety of conditions (see section 2.3.3 hereafter), among which structural integrity of the inlet structure and its surrounding. Also here some of the previously mentioned modes of failure may apply.

2.3.3 Flood risk aspects

Flood risk is defined as the product of the probability of a flood and its corresponding damage. The flood risk in a particular hydrological system or catchment area is the result of all possible flooding scenarios their probability of occurrence and their associated impact on society. Hence, aspects in flood risk comprise of:
1. the probability of floods, and
2. damage aspects.

(i) Probability of floods
Floods might occur simply due to the fact that the hydraulic loads on the protection works are larger than the resistance of the flood defence system. In determining the probability density functions of hydraulic boundary conditions (or loads), possible correlations between these hydraulic boundaries conditions should be taken into account. Floods, however, might also occur due to the fact that particular elements of the protection works are weaker than anticipated. All in all one has to deal with a lot of uncertainties that might result in a probable flooding. All these uncertainties have to be properly accounted for in determining the probability of floods. Hereunder taking the proposed Dutch emergency storage areas as an example, these uncertainties are further discussed and elaborated.

In designing and using emergency storage areas many uncertainties play a part, the practitioner discovers many uncertainties that are unfamiliar to him. In the Dutch geographical situation with the dense population the emergency storage area will be relatively small. This means that only the top of the time flood wave can be stored. The consequence is that a weir can not be applied as an inlet work. A weir will start to overflow when the water level reaches it's crest level. In case of an extreme flood wave this will start far before the moment of maximal discharge. The emergency storage area could then well be completely filled when the maximum discharge arrives and the mitigating effect on the maximum water level will be zero. It is clear that for an emergency storage area, that can store only a fraction of the entire flood wave a gated inlet work, that can be opened on the right moment, is needed.

In addition to this it should also be closed when the area is filled to prevent water spilling over the downstream dike of the emergency storage area, which is otherwise a certainty in the Dutch river country that slopes to the sea in it's entirety. (All dikes separation different polders become primary water defences in this way.)
Apart from closing the inlet work cutting the river dike just upstream of the dike dividing two polders is a classical solution. Hidden weirs in the dikes near Tiel, Gorkum and Sliedrecht, served this purpose in the old days and may be even today when their application is remembered.

The following uncertainties are identified.

1. Uncertainty in the model of the area; area, contour lines, position of the phreatic plane,
2. Uncertainty in the discharge coefficient of the inlet work; geometry, change of flow direction from river to inlet, steepening river slope due to discharge into emergency storage area,
3. Uncertainty in the prediction of the value and the moment in time of the maximum flood,
4. Uncertainty of the timely decision making by authorities,
5. Uncertainty of the timely operation by local authorities,
6. Uncertainty in the proper functioning of the seldomly used inlet work,
7. Uncertainty if the inlet work can withstand the force of the flow after a long stagnant period; failure to close will lead to flooding of the bordering polders!,
8. Uncertainty in the timely closure of the inlet work,
9. Uncertainty in the time needed for evacuation of man and animal (2-3 days),
10. Uncertainty in the water retaining ability and the stability of the new unproven dikes around the emergency storage area,
11. Uncertainty in the water retaining ability and the stability of the new unproven dikes around the villages in the emergency storage area,
12. Uncertainty in the stable political acceptance of the emergency storage concept ex ante (identification of unexpected consequences of flooding e.g. flora and fauna),
13. Uncertainty in the stable political acceptance of the emergency storage concept ex post. Identification of unexpected but all to real consequences of flooding after the first application. “Never again !!”

At first sight an emergency storage area is effective because it reduces the downstream water level. A reduced water level makes however only overtopping less likely. The other mechanisms as seepage piping, liquefaction, slip circle stay as likely. One could argue that their likelihood even increases due to the longer duration of the flood wave that was topped off.

The water level depends apart from the discharge also on the bed geometry and roughness of the river. Anecdote has it that the last flood in the Jangtze a 1/25 year discharge caused a 1/250 year water level followed by a dike breach at Jiu jang City.

The bed roughness may also increase if a reforested bed is not properly grazed and maintained.

If a bifurcation is found downstream of the emergency storage area the uncertainty in the splitting of the discharge makes the effect rather uncertain.

Conclusion
Before the concept of emergency storage is applied in the densely populated and relatively small scale Netherlands one should be certain of the effects.
Due to the limited size of the storage area uncertainties can easily compromise the effectiveness. To get the picture right all uncertainties should be modelled in order to study their effect and the extra area that is needed to give storage it’s required effect.

(ii) Damage aspects
Floods may result in the loss of live-stock and economic values, such as: damaged houses, cars, crops, commercial and industrial facilities, (hydraulic) infrastructure and so on. Even worse human lives might be lost. Further on environmental damage might occur as result of deposition of contaminated silt and toxic agents that were earlier picked-up by the flow from either the river bed or (protected) toxic storage depots.

The degree of flood-caused damage strongly depends on the number of inhabitants, the live-stock, the economic values invested in the catchment area as well as on the magnitude and origin of the floods. The magnitude of the floods is directly related to magnitude of flow velocities (i.e. hydraulic load on houses and other obstacles to the flow) and resulting inundation depths. The origin of the flood has consequences for the available time for evacuating inhabitants and live-stock as well as safeguarding economic goods and sealing off depots containing toxic agents. Except for flash floods, river floods usually offer sufficient time for evacuation and damage reduction measures. However, floods due to local downpours (rainfall) and storm surges at sea in general offer only limited time for taking these type of measures. Further on for sea-induced floods yields that the inundated flood water might be saline inducing more damage than fresh flood water.

2.3.4 Societal and institutional aspects

Both in policy making and during crisis situations the concept of system behaviour will have influence on decision making. It will also affect the communication on flood risk.

Policy making:

The current Dutch regulation is based on the analysis of single sections of dike stretches. Currently in the Floris project, the possibility of safety analysis based on dike-rings is investigated. A dike-ring can be defined as an area surrounded by dikes or higher grounds that must be safeguarded by the system of these flood defences against flooding. The inclusion of the interactions between the dike-rings, i.e. of system behaviour, is another further step in flood management policy. Currently responsibilities are attributed to the local government bodies (such as water boards and provinces). Also the effects of system behaviour will raise new issues in the decision making process. Consider the following example: authorities responsible for a dike-ring situated upstream plan dike strengthening in their area. These plans will also have implications for the safety levels of the dike-rings situated downstream and therefore require involvement of the downstream authorities in the decision making processes. It has to be noted that in the current plans, such as “room for rivers”, already the whole river system is considered as a whole and that, therefore, system behaviour is (implicitly) included to some extent in the decision making. Furthermore, safety assessment of the dikes and maintenance of safety levels will become complex when the interactions between the dike-rings have to be taken into account.

The current application of cost benefit analysis in flood protection is mainly limited to the dike-ring level. When system behaviour is included costs and benefits might consider different areas, and the whole river system should then be taken into account.

Crisis situations:

Furthermore active use of system behaviour in the river system during crisis situations will have implications. For example the use of emergency retention areas (noodoverloopgebieden) will require a different organisation of the decision making process. A study has been undertaken on the use of these areas when extremely high discharges occur [Scholtes, 2002], in which guidelines for the decision making process are proposed. The final decision will involve many parties and will have to be taken under time stress and immense pressure, while limited information is available. A recent study has been carried out to analyse the required information in these situations [Muller, 2003]. Some examples of important aspects are: the evacuation of the retention area, the available flood forecasts, the right moment of using the retention area. While decisions on evacuation are currently taken by local decision makers (e.g. the mayor), active use of system behaviour will probably shift this decision to a higher level.

Also other emergency measures will be related to system behaviour. The decision to heighten the dikes temporarily on a certain location with emergency measures (for example with sandbags), may result in a flood downstream and will thus “artificially” influence system behaviour.

Communication:

Communication on flood protection to society and it’s citizen will become more complex. How to explain the complicated interactions in the river system to non experts? Again, an observation has to be added. The plans developed for the installation of emergency retention areas (noodoverloopgebieden) show that it seems in some cases local interests can be overruled by the general interest of the country and the protection of the (more valuable) areas situated downstream.

Concluding remarks:

In the current standards for flood protection some kind of system behaviour is already taken into account in the prescribed design water levels. The FLORIS project will provide insight in flooding probabilities as well as consequences. Within probability assessment of the dike-rings, however, the safety of each dike-ring is considered separately.

In search for a sustainable development of flood defence several studies have been set up, which focus on solutions such as “space for rivers”, “green rivers” and “living with floods”. Many measures are proposed, but no real insight exists in the behaviour of the river system. Furthermore it is often stated without proper argumentation that “dike heightening is no realistic option”. However, consideration of new flood defence strategies requires insight in costs and benefits of all possible solutions without exclusion of certain types, as is also concluded by the Netherlands Bureau for Economic Policy analysis (CPB) [Verrips, 2001] . Insight in the system behaviour of the river system is an important element in the analysis of the effects of various risk reducing measures. These insights will provide a basis for new institutional structures which take into account system behaviour.

3 The Conceptual Framework

It was conceived necessary to develop a *generic* Conceptual Framework consisting of:

1. a Computational Framework for determining flood risk, and
2. an Institutional Framework taking care of societal as well as policies and decision making aspects

It is to be mentioned that most efforts were paid to the elaboration of the Computational Framework. Therefore no outline of the Institutional Framework is given in this chapter.

3.1 Needs and requirements for a *generic* Conceptual Framework

This *generic* Conceptual Framework should allow for all kind of safety-improvement measures to be considered simultaneously for the entire catchment area, while all aspects and processes relevant for studying effects of system behaviour on flood risk can be taken into account. The reasons (see also Chapter 2) here fore are:

- In assessing the consequences of safety-improvement measures not only their effects for a particular area but their effects on the safety of the entire catchment area are to be considered,
- The effect a particular safety-improvement measure might be mitigated by another safety-improvement measure. This implies that it is necessary to assess the effect of several safety-improvement measures simultaneously,
- Hydraulic/hydrological characteristics of the catchment area, geotechnical and constructional failure mechanisms, the uncertainties with respect to hydraulic loads (or boundary conditions) and the strength of protection works, and governing institutional procedures finally determine the effect of safety-improvement measures. Several of these aspects and processes are mutually interrelated. For instance, an initial breach in a dike-section might occur after this dike-section has been overtopped during a certain period of time, thereafter this initial breach might develop into a dike-breach. The final volume of water that flows through such dike-breach depends on the actual (erosion)strength of the dike and the hydraulic conditions occurring in front of the dike-breach. These hydraulic conditions on their turn are also governed by other eventualities that might happen in the catchment areas, such as other dike-breaches, the opening of green rivers and so on. Concluding it might be necessary to take into account several aspects and processes simultaneously,

3.2 The Computational Framework

The *aim* with respect to the Computational Framework would be the development of modular catchment-wide model/instrument. With modular is referred to the fact that it should be easy to change the hydraulic/hydrological characteristics of the catchment area, the considered failure mechanisms, the uncertainties in model parameters, and operation procedures for green rivers and retention areas. For instance in case it is considered to construct a compartment-dike in a particular polder, this would result in incorporating this dike in the existing hydraulic model schematization, adding of relevant failure mechanisms and uncertainties related to the strength of the compartment-dike. Thereafter the model/instrument is to be run and the consequences for the catchment-safety against flood is to be assessed. It might be obvious that the operation of such model/instrument requires expertise in the field of hydraulics/hydrology, geotechnical and constructional aspects, uncertainties as well operational control procedures. In addition one can not just develop such model for answering one particular question. Such model should be maintained by a central organisation, that is in charge of regularly updating this model to changes made in the catchment area.

Due to practical limitations and the current state of required technology/knowledge (see sections 3.2.1 to 3.2.4), simplifications for conducting the two case studies (see Chapter 4) were required. Nevertheless, a significant step towards such Computational Framework was made. Hereunder the various tools that are to be incorporated into the Computational Framework are discussed.

3.2.1 Hydraulic/hydrological model

The required flood modelling can be done using the SOBEK-Urban/Rural software package, developed at WL | Delft Hydraulics. This programme is capable of simulating the inundation of initially dry land, which is a prerequisite for establishing the effects of local dike failure on the safety of upstream and downstream located areas. Local rainfall can also be taken into account by the model. Triggers and controllers can be assigned to the model, enabling a dike- or structure failure to be initiated based on the actual value of a particular hydraulic parameter (water level, discharge, pressure difference etc.). Further on it is possible to program triggers and controllers in Matlab and combine these triggers and controllers to the hydraulic model. In this way any kind of failure mechanism (for instance including soil stability calculations) can be incorporated into the model. Dike-breach development can be pre-defined as a relative function of time. Further on recently it became possible to define dike-breach development as function of dike strength parameters and the actual hydraulic conditions near the dike-breach location (i.e. Verheij–vdKnaap(2002) breach formula, see Appendix B). One of the current constraints in modelling large catchment area’s is the amount of computational time required for making the hydraulic simulations. With the computational power of future generation of computers, this constraint may become less important.

3.2.2 Water quality model

In flooding also the spread of saline flood water, contaminated silt and toxic agents is to be considered. Recently at WL | Delft Hydraulics a two dimensional water quality (2D WQ) model was developed enabling the computation of the spread of contaminated silt and toxic agents. This 2D WQ model can be combined with the above discussed SOBEK-Urban/Rural software package. At present it is considered to make this 2D WQ model available to all SOBEK-Urban/Rural users. The 2D WQ model was developed in the framework of an another Delft Cluster project, being DC 02.03.03: Impacts of flooding.

Obviously water quality aspects as discussed above are of importance in assessing flood damage and hence flood risk (see also section 2.3.3). This makes the framework for computational prediction of flood risk and making decisions on how to control and mitigate flood risk even more complex. The objective of the present study is to make a first step, i.e. to achieve a good understanding of all water-quantity related processes involved in system behaviour and to develop a basic concept for flood risk prediction. Taking this into account water-quality related aspects were not further elaborated.

3.2.3 Geotechnical and structural tools/models

Computation models for geotechnical dike failure reflect the effects of key phenomena which play a role in the various mechanisms of failure. For comprehensive and up to date descriptions of computation models for geotechnical failure reference is made to [TAW 2001], the Technical Report on Water Retaining Earth Structures, issued by the Dutch Technical Advisory Committee on Flood protection. Generally the key phenomena may be characterized as:

- Dependency on potential head difference between river and (ground) water levels in the protected area near the dike.
- Time dependency, related to delay effects (e.g. non stationary pore pressure development)
- Time dependency, related to duration of processes (e.g. infiltration or erosion)
- Uncertainties of strengths, among which residual strength after initial failure.

As already stipulated, occurrence of geotechnical failure mechanisms highly depends on development of pore pressures inside the dike or in the soil strata below the dike. This process is driven by potential head fall between river and inland (ground)water table, and depends on the geohydrological configuration and boundary conditions. It may be interfered by autonomous processes, e.g. ongoing consolidation of substrata in case of recent aggravation of the dike or, most common, infiltration due to rainfall. However, the effects of such processes is usually not explicitly taken into account in geotechnical analyses. Actual pore pressures in the dike or the subsoil at some time reflect delayed response to the time history of the river head level. Consequently the probabilities of occurrence of geotechnical failure modes depend on this phenomenon, although not all of the mechanisms to the same extent.

Time duration of processes plays an important role when considering mechanisms like erosion of inner slope revetment due to overtopping or failure of the inner slope due to infiltration of overtopping water and, probably most typical, erosion of the dike body after the occurrence of an initial failure mechanism. However, knowledge of time duration of these processes is very poor, because it has hardly been considered as a research topic of interest until recently. This reflects the (still current) classical philosophy that a design of a water retaining structure should be able to withstand design loads without initial failure, preferably regardless the duration of such loads. For flood risk analysis, though, the sequence of triggered damage mechanisms after initial failure, finally leading to breach is essential, because it may dominate the probability of dike breach.

3.2.4 Flood risk tools

Let us consider an analysis in which also more than one dike-ring is involved. In such an analysis collapse of a dike-ring may have influence on the water levels for other dike-rings and hence also on the failure probability and risks. A complete system analysis considers all the dike-rings within a hydrological area and reveals which flood scenario's can take place and evaluates their probabilities and impacts.

The general expression for such a flood-risk calculation is given by:

$$R = E(D) = \int_{t=0}^{\infty}\int D(\underline{x},t)\, f(\underline{x},t)\, d\underline{x}\ dt \qquad (3.1)$$

Provided that the integral exists. In Equation (3.1) is:

R	risk
E(..)	expectation
t	is the time
$\underline{x}$	the vector with all the stochastic parameters which play a role in the problem (river discharge, wind speed, sea level, soil properties, dike lining, emergency measures, polder roughness, behaviour of secondary dams, etc.)
D	is the damage capital (dependent on location and moment of flooding)
$f(\underline{x},t)$	is the joint probability density function of $\underline{x}$ which may be a function of t.

Note that the flood damage may depend on the specific circumstances and conditions (i.e. specific realizations of the random parameters $\underline{x}$) which lead to dike breach and the specific dike breach locations. Earlier flood risk analyses in the Netherlands involved only one protected area (dike ring area). In those analyses often the simplification has been adopted that the flood damage is more or less independent of the circumstances that lead to dike breach, with the obvious exception of the location of dike breach. In such cases it suffices to determine the (location dependent) probabilities of dike breach and flood damages separately (see e.g. PICASO), thus yielding a significant reduction of computational burden to determine flood risk. However, in the case of multiple areas in which the system response of the river significantly affects the development of flood damages, this simplification cannot be justified any longer. This implies the necessity of full evaluation of the integral in equation (3.1). The numerical approach adopted in this project is a Monte Carlo type procedure (see section 4.3), in which actual flood damage is evaluated in each and every simulation run (in which dike breach occurs), thus providing the basis for full evaluation of the integral in equation (3.1).

In the Netherlands, the assessments for the damage (D) are made using the so-called "HIS-Schade en Slachtoffer Module (HIS-SSM)". Using this method the anticipated economic damage and number of casualties due to a flooding can be established. In establishing possible damage use is made of GIS oriented databases. Using these databases, for a arbitrary area in the Netherlands an inventory can be made of the number of inhabitants, houses, cars, agricultural land and so on. In computing the anticipated damage and number of casualties for each area following (hydraulic) parameters are needed, being: the speed in which water levels rise, the maximum inundation depths, the maximum flow velocities, a shelter factor, and the occurrence of wind-induced waves. For more information on the HIS-SMM model, reference is made to Vrisou van Eck, Huizing and Dijkman (2002) and to Vrisou van Eck, Kok and Vrouwenvelder (2000).

At present WL | Delft Hydraulics in close consultation with $HKV_{Lijn\ in\ water}$ develops a generic damage assessment tool for making damage assessments abroad. For more information, reference is made to Karin Stone of WL | Delft Hydraulics. Presently no tools are available for systematically assessing environmental damage due to the spread of contaminated silt and toxic agents during flooding.

4 Description of Case studies

4.1 Scope and aim of the case studies

In total two case studies were conducted:

1. Case study no. 1, concerning a catchment area having only one river branch and two polders, respectively located on the Northern and the Southern river bank, and
2. Case study no. 2, having a catchment area similar to the catchment of case study no. 1 with the notation that an upstream located emergency retention polder was added on the Southern river bank.

The reason for conducting both case studies was *primarily* for evaluating the performance of the Computational Framework (see section 3.2) in estimating flood risk in catchment areas where effects of system behaviour are of importance. Taking this aim into account it was decided to start with hypothetical but realistic case studies instead of modelling some specific real part of the Dutch river delta. In this way a lot of effort in collecting data as well as computational effort was saved.

4.2 Catchment- and River Characteristics

4.2.1 Model description of Case study no 1

(i) Model schematization

Figure 4.1 depicts in a schematic way the catchment considered in Case study no 1. This catchment comprise of one river branch only, a Northern polder of 16992 hectares and a Southern polder of 2880 hectares. Both polders are protected against flooding by dikes. These dikes may breach at predefined locations (see Fig 4.1), respectively at three locations (L1, L2 and L3) along the Northern polder and at one location (i.e. L4) along the Southern polder.

The river is considered to have a prismatic rectangular cross-sectional profile and a constant bed-level slope of 10 cm per kilometre (i.e. 10^{-4}) towards the West. The rectangular cross-sectional profile has a width of 500 m and a Nikuradse bed-roughness value of k_n = 0.05. In the SOBEK-Urban/Rural hydrodynamic computations, the flow in the river was considered as one-dimensional flow and computed with a spatial discretization of 100 m.
The polders have a constant terrain slope of 10 cm per kilometre (i.e. 10^{-4}) towards the West. In the polders there is no terrain slope perpendicular to the river. Although a main road is discerned in the Northern polder, for all polders yields that there are no vertical-line objects that might serve as a water-barrier. The dikes surrounding the polders have an elevation of 4.65 meters above the adjacent local polder terrain level, which is just above a water level corresponding to a discharge having a return period of 90 years (i.e. Q_{peak}=9000 m^3/s, see

Table 4.5). The land-use in each polder is depicted in Figure 4.2. The land-use and corresponding areas of each polder are given in Table 4.1. Assuming that floods occur in winter, for the polders Nikuradse (k_n) roughness values were selected for the surface-roughness on basis of the actual land-use. In Table 4.2 the Nikuradse (k_n) roughness values for each land-use type are given. In the SOBEK-Urban/Rural hydrodynamic computations, the flow in the polders was considered as two-dimensional flow and computed with spatial discretization both in x- and y- direction of 400 m.

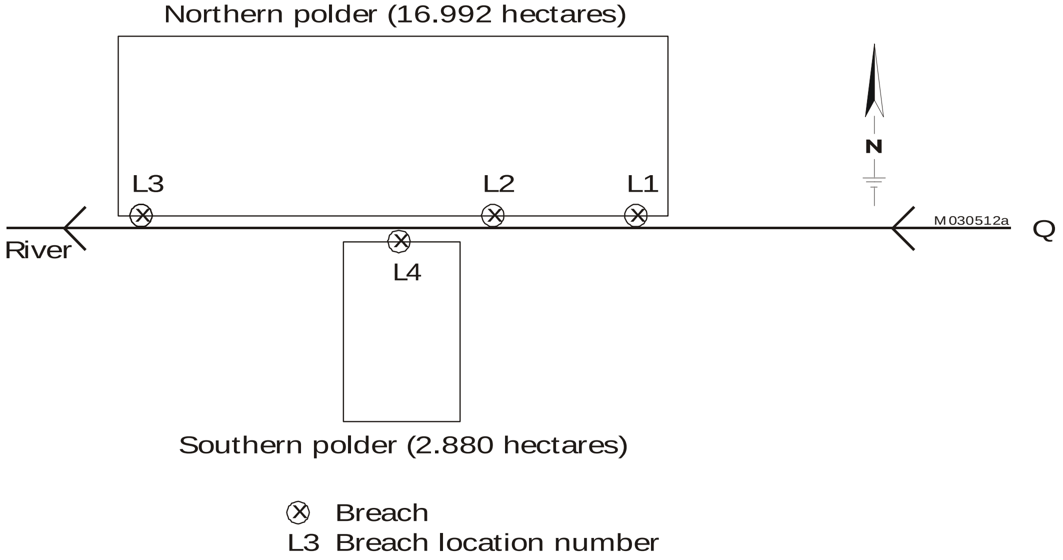

Fig. 4.1 Schematic model schematization of Case study no 1

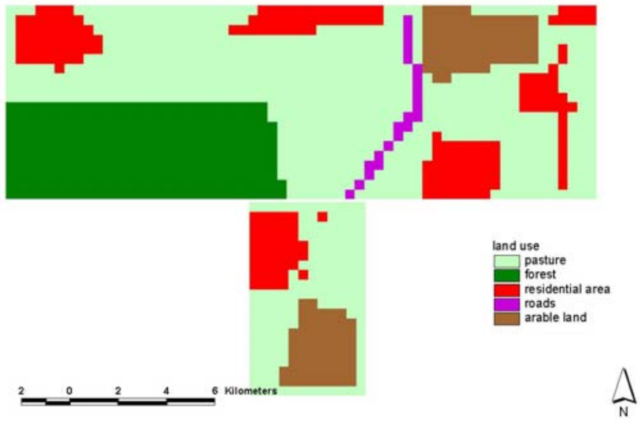

Fig. 4.2. Land-use in Northern and Southern polder of Case study no 1

Table 4.1 Land-use and corresponding areas for polders in Case study no 1

Area	**Pastures** *hectares*	**Forest** *hectares*	**Residential Areas** *hectares*	**Roads** *hectares*	**Arable land** *hectares*	**Total** *hectares*
Northern polder	9488	3872	2192	336	1104	**16992**
Southern polder	1504	-	464	-	912	**2880**

Table 4.2 Land use and applied Nikuradse bed roughness (k_n) values

No.	Land-use Type	Nikuradse roughness (k_n)
1	Pastures	0.3
2	Forest	10
3	Residential areas	10
4	Roads	0.1
5	Arable land (crops)	0.1

(ii) Boundary conditions

The boundary conditions comprise of an upstream discharge hydrograph and a downstream stage-discharge relationship. As an example an upstream discharge hydrograph with a return period of 90 years and having a peak discharge of 9000 m^3/s is depicted in Fig 4.3. The stage-discharge relationship was constructed in such way that all along the entire river length uniform flow conditions (i.e. surface-water-level slope equal to bed-level slope) prevail for constant upstream discharges.

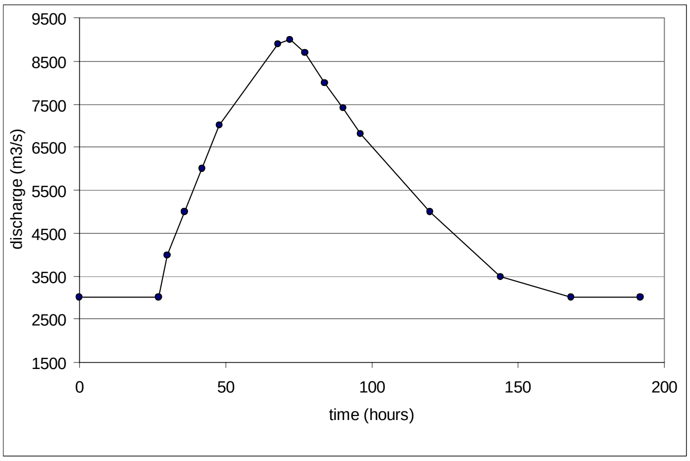

Fig. 4.3. Upstream discharge hydrograph with a return period of 90 years.

4.2.2 Model description of Case study no 2

(i) Model schematization

Figure 4.4 depicts in a schematic way the catchment considered in Case study no 2. This catchment consist of one river branch only, a Northern polder of 16992 hectares, a Southern polder of 3456 hectares and an Emergency retention polder of 4896 hectares. All polders are protected against flooding by dikes. These dikes may breach at predefined locations (see Fig 4.4), respectively: at three locations (L1, L2 and L3) along the Northern polder; at one location (i.e. L4) along the Southern polder; and two locations (i.e. L5 and L6) along the Emergency retention polder.

For Case study no 2 yields that the catchment (i.e. river and polders) schematization is identical to the one of Case study no 1, with the notation that an upstream located emergency retention polder was added and that the total area and land-use in the Southern polder are slightly different. The land-use and corresponding areas of each polder are given in Table 4.3.

(ii) Boundary conditions

The boundary conditions applied in Case study no 2 are identical to the ones applied in Case study no 1.

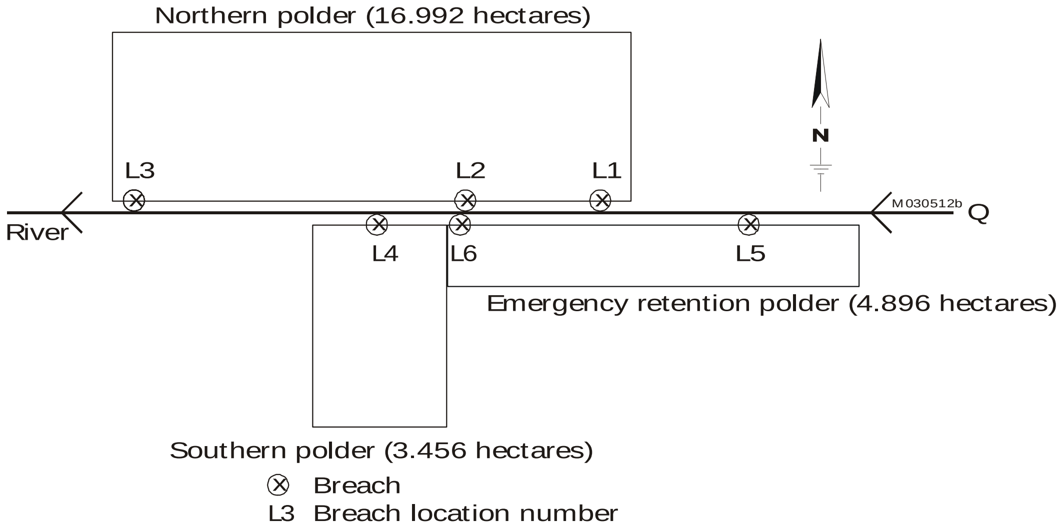

Fig. 4.4 Schematic model schematization of Case study no 2

Table 4.3 Land-use and corresponding areas for polders in Case study no 2

Area	**Pastures** *hectares*	**Forest** *hectares*	**Residential Areas** *hectares*	**Road** *hectares*	**Arable land** *hectares*	**Total** *hectares*
Northern polder	9488	3872	2192	336	1104	**16992**
Southern polder	1968	-	576	-	912	**3456**
Emergency retention polder	4896	-	-	-	-	**4896**

4.3 Applied method in computing flood risk

As mentioned in section 3.2.4 it is necessary to use a simplified approach for determining flood risk. Therefore, in the both case studies the flood risk was computed as explained below.

Step 1. Find the failure probabilities for each dike-section without interaction

In the usual way (for instance by application of the computer code “PC-ring”), for each dike-ring, the failure probability for each section and each failure mechanism is calculated. Also the system probability P(F) of having failure “somewhere in the whole system” is determined. However, only statistical and no physical system interactions are taken into account. The dike-sections with the highest failure probability are selected as possible dike-breach locations and mean “design values” for considered failure mechanisms are determined on the basis of a first order reliability (FORM)-analysis.

FORM is a relatively fast calculation procedure that is based on a transformation to normal variables and a linearization of the failure boundary. The point selected to perform the linearization, the so called design point, is the point in the failure domain with the highest probability density. For more details information reference is made to TAW-CUR Report 141 (CUR/TAW, 1990) or the TAW guideline "Fundamentals on Water Defences (TAW, 1998).

Step 2. Monte Carlo Analysis

In a (Crude) Monte Carlo analysis, first the actual values for the parameters of the in step 1 considered failure mechanisms at the selected breach locations are determined. These actual values are determined by sampling probability density functions f(x), with means equal to the in step 1 mentioned mean “design values” and standard deviations selected on basis of expert judgement. In addition the (Crude) Monte Carlo analysis includes sampling on probability density functions for system boundary conditions. The sampling results in a number of scenario’s (or runs) of which the possible consequences of failure are to be established by making hydraulic computations using the sampled (or actual) values, while taking into account the various considered failure mechanisms. In making these hydraulic computations all possible physical interactions are automatically taking into account. The final result of the (Crude) Monte Carlo analysis is a series of computed scenario’s in which flooding might have occurred due to one or several dike breaches and/or overflow. The flood damage of each computed scenario can be determined afterwards (see section 4.5).

Step 3. Determination of the risk

In the present case studies, a (Crude) Monte Carlo, as described in Step 1 and 2 above, has been used. An estimator of the flood risk in Equation (3.1) is:

$$R = \frac{1}{N}\sum_{i=1}^{N} D_i \qquad (4.1)$$

where:
R = Flood risk for the period of one year,
N = Number of Monte Carlo runs (or computed scenario’s). The considered upstream discharge hydrograph represents a period of one year (see also section 4.4). Therefore, each Monte Carlo run represents a period of one year as well, and
D_i = Damage in run i (if a run does not lead to a failure, D_i is equal to zero).

The estimator (4.1) is unbiased and consistent provided that the numerical procedure to calculate D_i is physically correct**.** Meaning that the physics involved in the occurrence of a failure mechanism, the occurrence of an initial breach as well as the breach-growth development are to be correctly accounted for in a Monte Carlo run**.** This does not only yields for a Monte Carlo analysis, but yields for any other approach for determining flood risk. Fulfilment of this condition is not obvious. As argued in sections 6.2.3 and 6.3 still more knowledge on the above mentioned physical processes is required for improving their mathematical formulations in the Computational Framework, that was applied in making the Monte Carlo runs (or scenario’s). For the mathematical formulations of these physical processes as applied in both case studies, reference is made to section 4.4.

4.4 Description of stochastic model parameters (Failure mechanisms, Dike-breach growth and Flood wave)

In the process of a dike-breach a distinction was made between:

1. The occurrence of an initial breach in a dike due to a particular failure mechanism, and
2. The growth of the initial breach as a result of the erosive flow passing through the initial breach.

As mentioned before, the dikes surrounding the polders can only breach at predefined breach locations (see Fig 4.1 and Fig 4.4).

Ad 1) Both in Case study no 1 and Case study no 2 only the overtopping- and piping failure mechanisms were considered. Failure due to overtopping is considered to occur instantaneously in case the water level in the river exceeds the critical water level for overtopping (i.e. the local crest level of the dike). Failure due to piping occurs when the critical water depth for piping is exceeded over a Critical duration for exceedance of piping depth. In other words the dike will fail due to piping when a critical head difference across the dike is continuously exceeded longer than some critical time lapse. The underlined definitions are the stochastic model parameters considered (see Table 4.5). The rules applied in SOBEK-Urban/Rural for determining if a failure due to overtopping or piping might occur are given in section C.1 of Appendix C.

Ad 2) Both in Case study no 1 and Case study no 2, two phases are discerned in the dike-breach growth process. During the first phase, for a constant width of 20 metres the dike is lowered from its initial elevation to the level of the adjacent polder area. During the second phase the dike-breach grows linearly in width only over a time-period T_{breach} (or Duration of breach growth) till a final breach width is attained. The underlined definitions are the stochastic model parameters considered (see Table 4.5).

Another uncertainty in model parameters that was taken into consideration in Case study no 1 as well as in Case study no 2, was the magnitude of the upstream peak discharge. This uncertainty was taken into account by introducing a multiplication factor (or Discharge scaling factor) to the peak discharge (Q_{peak} = 9000 m^3/s) of the upstream discharge hydrograph having a return period of 90 years (see Fig. 4.3). The time basis of the discharge hydrograph was not considered as a stochastic model parameter and represents the period of one year. Return periods for upstream discharge are given in Table 4.5. These return periods are according to the probability density function for the discharge scale factor given in Table 4.4.

Resuming both in Case study no 1 and Case study no 2, following six stochastic model parameters were considered, viz: the critical water level for overtopping; the critical water depth for piping; the critical duration for exceedance of piping depth; the duration of breach growth; the final breach width; and the discharge scaling factor. The first five stochastic parameters yield for each and every dike-breach location, unless stated otherwise. The selected probability density functions (including mean, standard deviation or coefficient of variation) of these six stochastic parameters are given in Table 4.4. For more information on how these probability density functions were derived, reference is made to section 4.3. In addition for more information on the probability density functions, reference is made to section C.2 of Appendix C.

Table 4.4 Probability density functions for considered stochastic model parameters in Case study no 1 and Case study no 2.

Stochastic model parameter	**Probability density function**	**Mean**	**Coefficient of Variation (V) or Standard deviation (σ)**
Critical water level for overtopping (m)	Normal	12.12 [1)]	σ = 0.3
Critical water depth for piping (m)	Lognormal	4	V = 0.125
Critical duration for exceedence of piping depth (hrs)	Lognormal	6	V = 0.33
Duration of breach growth (hrs)	Lognormal	40	V= 0.5
Final breach width (m)	Lognormal	100	V = 0.5
Discharge scaling factor (-)	Exponential	0.3	σ = 0.2

Notes:

1. The critical water level for overtopping varies per dike-breach location and corresponds to a discharge with a return period of 90 years (see also Table 4.5)
2. Coefficient of variation is equal to mean divided by the standard deviation

Table 4.5 Return periods for upstream discharges

Return Period *years*	**Discharge** *m^3/s*
47	7840
70	8550
90	9000
260	10900
477	12000

4.5 Computing Flood-caused Damage

Using the computed maximum water depths and the Dutch Standard Damage model (i.e. HIS-SSM, see section 3.2.4), the flood-caused damage in each considered flood scenario was established. These damages are used in Eq 4.1 for establishing the flood risk (see section 4.3).

Establishing damage using the Dutch Standard Damage model was done as follows:

1. The computed maximum water depths for each computational grid cell are classified in accordance with the water depth classes shown in Fig 4.5,
2. For each grid cell with grid cell-size of 400 m, the actual damage is computed by multiplying its maximum potential damage value (depends on its land-use type, see Table 4.6) with its actual damage factor that depends on the maximum water depth class and can be derived from Fig 4.5,
3. The resulting damage in each polder is established by the summation of the damage of each individual grid cell.

In both case studies only economic damage was considered.

For Case study no 1 and Case study no 2, the potential damage (i.e. damage factor equal to unity) for each polder is respectively given in Tables 4.7 and 4.8.

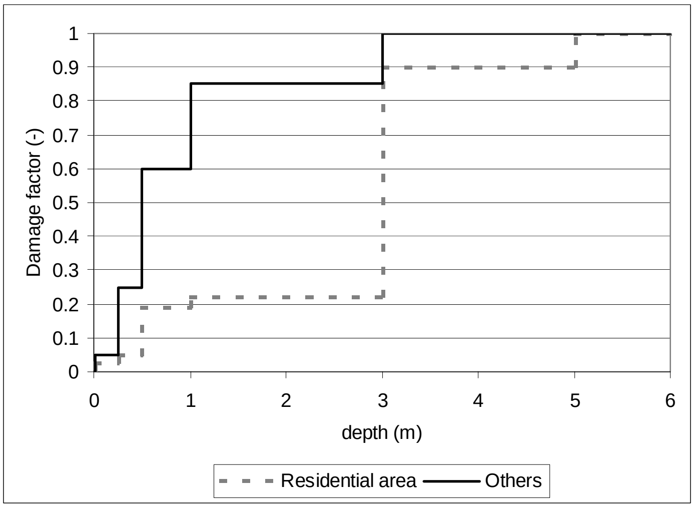

Figure 4.5 Damage factors for residential areas and other type of land-use. Other type of land-use refers to pastures, forest, roads and arable land.

Table 4.6 Maximum possible damage per land-use type in 10^3 Euro per hectare

No.	Land-use Type	Maximum Damage in 10^3 Euro per hectare
1	Pastures	15.9
2	Forest	0.9
3	Residential areas	5681.8
4	Roads	5.7
5	Arable land (crops)	15.9

Table 4.7 Case study no 1, Potential flood damage in 10^6 Euro (i.e. damage factor = 1)

Area	Pastures	Forest	Residential Areas	Roads	Arable land	Total
	10^6 Euro	*10^6 Euro*	*10^6 Euro*	*10^6 Euro*	*10^6 Euro*	*10^6 Euro*
Northern polder	150.9	3.5	12454.5	1.9	17.6	**12628.5**
Southern polder	23.9	-	2636.4	-	14.5	**2674.8**

Table 4.8 Case study no 1, Potential flood damage in 10^6 Euro (i.e. damage factor = 1)

Area	Pastures	Forest	Residential Areas	Road	Arable land	Total
	10^6 Euro	*10^6 Euro*	*10^6 Euro*	*10^6 Euro*	*10^6 Euro*	*10^6 Euro*
Northern polder	150.9	3.5	12454.5	1.9	17.6	**12628.5**
Southern polder	31.3	-	3272.7	-	14.5	**3318.5**
Emergency retention polder	77.9	-	-	-	-	**77.9**

5 Results of Case studies

In this Chapter the results of Case study no. 1 and Case study no 2 are respectively given in sections 5.1. and 5.2. Concluding remarks are given in section 5.3. For a description of both case studies, reference is made to Chapter 4.

5.1 Results of Case study no 1

Case study no 1 concerns two polders only, respectively a Southern polder and a Northern polder. In total four breaching locations were considered (see Fig 4.1), respectively three locations (L1, L2 and L3) along the Northern polder and one location (i.e. L4) along the Southern polder. For all these breaching locations the same stochastic model parameters were applied.

In order to be able to distinguish positive or negative effects of system behaviour on flood risk, simulations were made considering the Southern polder only, considering the Northern polder only, and considering both polders (i.e. the entire catchment). Further on three different options were discerned, viz:

1. *Option 1*: Considering both the overtopping and piping failure mechanisms. In Table 5.1 the corresponding flood risks on basis of 3000 runs (or scenario's) are given.
2. *Option 2*: Considering the piping failure mechanism only. In Table 5.2 the corresponding flood risk on basis of 3000 runs (or scenario's) are given, and
3. *Option 3*: Considering the overtopping failure mechanism only. In Table 5.3 the corresponding flood risk on basis of 1000 runs (or scenario's) are given.

For the corresponding flood-caused damages per scenario, reference is made to Appendix D.

From Tables 5.1 to 5.3 following observations and conclusions can be made:

1. The number of failures (i.e. when flooding occurs) for the Southern polder reduces considerably when system behaviour is taken into account, compared to an analysis considering the Southern polder only (i.e. without taking system behaviour into account). This phenomena is irrespective of the type of failure mechanisms considered, viz: (i.e. Table 5.1: 28 < 42 failures; Table 5.2: 11 < 20 failures; and Table 5.3: 13 < 17 failures).
2. The number of failures for the Northern polder does not reduce when system behaviour is taken into account (see Tables 5.1 to 5.3). This is due to the geographical orientation of the Northern polder, more precisely the Northern polder has two dike-sections located far more upstream than the Southern polder (see Fig 4.1). For this reason the probability of failure for the Northern polder is larger than for the Southern polder. This phenomena is due to the fact that once failure at breach locations L1 or L2 occurs, the hydraulic load near breach location L4 will reduce, and as a result the probability of the flooding of the Southern polder will reduce. Concluding: for single rivers (i.e. having one river branch only) and flood-waves dominated by upstream discharges only, yields that for the same protection standards (i.e. the same stochastic model parameters were applied at each

breach location) downstream located polders benefit more from effects of system behaviour than the upstream located polders.

3. The calculated flood risk reduces when system behaviour is taken into account, compared to an analysis considering each polder individually (i.e. without taking system behaviour into account). System behaviour has a positive effect on the overall flood risk in the catchment area irrespective of the type of failure mechanisms considered. In other words, the flood risk in the entire catchment is less than the summation of the flood risks when considering each polder individually (i.e. Table 5.1: 69.8 M€ < 20.2 M€ + 61.4 M€; Table 5.2: 59.4 M€ < 7.6 M€ + 55.6 M€; and Table 5.3: 89.7 M€ < 31.4 M€ + 77.0 M€),
4. The flood risk in the catchment due to considering a set of failure mechanisms should be equal or larger than the flood risk due to considering only one of these failure mechanisms. The fact that the flood risk in the catchment due to both the overtopping and piping failure mechanism amounts to 69.8 million Euro (see Table 5.1; 3000 scenario's) and the flood risk due to the piping failure mechanism only amounts to 59.4 million Euro (see Table 5.2; 3000 scenario's) is in accordance herewith,
5. In computing flood risk a sufficient number of flood scenario's are to be considered in the Monte Carlo analysis. This is illustrated by the fact that the flood risk in the catchment based on 3000 scenario's and considering both the overtopping and piping failure mechanisms amounts to 69.8 M€ (see Table 5.1), which is less than the flood risk of 89.7 M€ (see Table 5.3) based on 1000 scenario's and considering the overtopping failure mechanism only. The reason for this anomaly (see also point 4 above) is the fact that in the first 1000 scenario's more damage occurred as in the succeeding 2000 scenario's. In Fig 6.1 for considering both the overtopping and piping failure mechanisms, it can be seen that the flood risk based on 3000 scenario's is about 1.4 times smaller as the flood risk based on 1000 scenario's only. Applying this factor of 1.4 to the flood risks given in Table 5.3, results in a flood risk for the entire catchment of 64.1 M€ (=89.7 M€/1.4) due to the overtopping failure mechanism only, which is smaller than the flood risk of 69.8 M€ considering both the overtopping and piping failure mechanisms. For more detailed information on the required number of flood scenario's to be considered in the Monte Carlo analysis, reference is made to section 6.4.
6. In assessing flood risk all possible failure mechanisms are to be considered jointly. Adding floods risk due to separate failure mechanism is not correct, since the occurrence of various failure mechanism might be mutually interrelated. For instance, the overall risk in the catchment due to both the overtopping and failure mechanism should be equal or less than the summation of this flood risk due to only the piping failure mechanism and only the overtopping failure mechanism [i.e. 69.8 M€ (see Table 5.1) < 59.4 M€ (see Table 5.2) + 64.1 M€ (see Table 5.3 and applying factor of 1.4, see point 3 above)]. This is due to the fact that when a dike-section fails due to piping, no additional flood risk may be expected from other failure mechanisms than piping at the same location (see section 6.2.3 item (iii)).

Table 5.1. Case study no 1, Option 1, Number of Failures and Flood risk in 10^6 Euro on basis of 3000 runs (or scenario's, see also Table D.2 in Appendix D), considering the overtopping and piping failure mechanisms and probability density functions for model parameters as given in Table 4.4. Flood risks for considering each polder individually and for considering the entire catchment are given.

Number of Failures (Overtopping and piping, 3000 scenario's)			
Area considered	*Southern polder*	*Northern polder*	*Entire catchment*
Northern polder only	-	59	-
Southern polder only	42	-	-
Entire catchment	28	59	60
Flood risk in 10^6 Euro (Overtopping and piping, 3000 scenario's)			
Area considered	*Southern polder*	*Northern polder*	*Entire catchment*
Northern polder only	-	61.4	-
Southern polder only	20.2	-	-
Entire catchment	8.9	60.9	69.8

Table 5.2 Case study no 1, Option 2, Number of Failures and Flood risk in 10^6 Euro on basis of 3000 runs (or scenario's, see also Table D.3 in Appendix D), considering only the piping failure mechanism and probability density functions for model parameters as given in Table 4.4, except for model parameter Critical water level for overtopping". Flood risks for considering each polder individually and for considering the entire catchment are given.

Number of Failures (Piping only, 3000 scenario's)			
Area considered	*Southern polder*	*Northern polder*	*Entire catchment*
Northern polder only	-	54	-
Southern polder only	20	-	-
Entire catchment	11	54	55
Flood risk in 10^6 Euro (Piping only, 3000 scenario's)			
Area considered	*Southern polder*	*Northern polder*	*Entire catchment*
Northern polder only	-	55.6	-
Southern polder only	7.6	-	-
Entire catchment	4.1	55.3	59.4

Table 5.3 Case study no 1, Option 3, Number of Failures and Flood risk in 10^6 Euro on basis of 1000 runs (or scenario's, see also Table D.4 in Appendix D), considering only the overtopping failure mechanism and probability density functions for model parameters as given in Table 4.4, except for model parameters "Critical water depth for piping" and "Critical duration for exceedance of piping depth". Flood risks for considering each polder individually and for considering the entire catchment are given.

Number of Failures (Overtopping only, 1000 scenario's)			
Area considered	*Southern polder*	*Northern polder*	*Entire catchment*
Northern polder only	-	19	-
Southern polder only	17	-	-
Entire catchment	13	19	19
Flood risk in 10^6 Euro (Overtopping only, 1000 scenario's)			
Area considered	*Southern polder*	*Northern polder*	*Entire catchment*
Northern polder only	-	77.0	-
Southern polder only	31.4	-	-
Entire catchment	13.7	76.0	89.7

5.2 Results of Case study no 2

Case study no 2 concerns the use of an emergency retention polder aiming at improving the safety of the downstream located Southern and Northern polder (see Fig 4.4). Two different options for operating the emergency retention polder were discerned. The effectiveness of the operation of the emergency retention polder in case of extreme discharges, is based on comparing flood risk in the catchment with and without controlled flooding.

The various sub-options discussed hereafter, comprise of a difference in the piping failure mechanism only. Hence the overtopping failure mechanism is always taken into account. The reason for doing so is the fact that the operation of the emergency retention polder aims at reducing the peak discharges near the dikes along the downstream located polders. By reducing these peak discharges the number of times that these dikes are overtopped will be reduced. Omitting the overtopping failure mechanism was, therefore, not considered.

5.2.1 Reference situation

The reference situation refers to the fact that the most upstream located polder is used as an ordinary polder (i.e. situation without controlled flooding). The dikes along this polder might breach at two locations. Hence in the reference situation in total six breaching locations were considered (see Fig 4.4), respectively three locations (L1, L2 and L3) along the Northern polder; one location (i.e. L4) along the Southern polder; and two locations (i.e. L5 and L6) along the most upstream located polder. For all these breaching locations the same stochastic model parameters were applied.

For the reference situation without controlled flooding two different sub-options were discerned, viz:

Ref a. Considering both the overtopping and the piping failure mechanisms, and

Ref b. Considering the overtopping failure mechanism only.

As explained in section 4.3, in the Monte Carlo analysis a number of different flood scenario's are determined by sampling the probability density functions of the considered stochastic model parameters (see also Table 4.4). The result of such sampling for reference situation no. a (i.e. Ref a) are given in Table E.1 of Appendix E.

5.2.2 Applied Operation of Emergency Retention Polder

Making the most upstream located polder suited to act as an emergency retention polder, implies that the dikes all along this polder (i.e. not only along the river) are to be strengthened to allow for overtopping and retaining of water. Taking this strengthening into account, it was considered that the safety of the dikes along the upstream emergency retention polder is much higher than the safety of the dikes along the Southern and Northern polder. Therefore, breaching of the dike along the emergency retention polder was not considered. Hence in the options were the most upstream located polder serves as an emergency retention polder, in total only four breaching locations were considered (see Fig 4.4), respectively three locations (L1, L2 and L3) along the Northern polder and one location (i.e. L4) along the Southern polder. For all these breaching location the same stochastic model parameters were applied.

In the various operation options discussed hereafter, the dikes along the emergency retention polder have crest-levels just above a water level corresponding to design discharges having return periods of 70 and 90 years. The storage capacity of the emergency retention polder for these crest-levels is given in Table 5.4. In addition in Table 5.4, the volume of water above such crest-level, that is contained by an upstream flood-wave having a return period of 260 years (Q = 10900 m^3/s) is given.

Table 5.4 Storage capacity of the emergency retention polder and volume of water that is contained in an upstream flood wave having return period of 260 years (Q = 10900 m^3/s) as function of crest-level of the dikes along the emergency retention polder.

Crest level Return period *years*	**Discharge** *m^3/s*	**Storage capacity of emergency retention polder** *10^6 m^3*	**Volume above crest level in 260 yrs (Q=10900 m^3/s) flood-wave** *10^6 m^3*
70	8550	213.0	213.0
90	9000	227.7	156.9

Two different options for operating the emergency retention polder were discerned.

1. Emergency retention polder equipped with a fixed weir,
2. Emergency retention polder equipped with an adjustable weir,

Ad 1) Option 1: Emergency retention polder equipped with a fixed weir

The entire dike along the river-side of the emergency retention polder is considered to act as a weir. Due to the fact that river flow was modelled as one-dimensional flow, such weir was schematized as a number of separate weirs, each having a crest width of 1000 m. It is considered that hydraulic consequences on computed flood risk due to this schematization are negligible. The possibility of dike-breaches in the dikes along the emergency retention polder was not considered. Two different sub-options were discerned, viz:

1a. Applying for the weir(s), a crest level corresponding to discharges having a return period of 70 years (peak discharge of 8550 m^3/s, see Table 4.5) and considering both the overtopping and piping failure mechanisms, and

1b. Equal to option 1a, except that only the overtopping failure mechanism is considered.

Ad 2) Option 2: ***Emergency retention polder equipped with an adjustable weir***

In this option the inflow of flood water in the emergency retention polder is controlled by an adjustable weir having a constant crest width of 700 m. The lowest possible crest-level of this adjustable weir is 2 metres below the crest level of the adjacent dikes, which correspond to a water level belonging to a discharge with a return period of 90 years (i.e. Q = 9000 m^3/s). The possibility of dike-breaches in the dikes along the emergency retention polder was not considered. Two different sub-options were discerned, viz:

2a. Controlling the weir for maintaining discharges downstream of the emergency retention polder below 8900 m^3/s (i.e. the design discharge for the downstream located Northern and Southern polders is equal to 9000 m^3/s) and considering both the overtopping and piping failure mechanisms, and
2b. Same as sub-option 2a, except that only the overtopping failure mechanism is considered.

As an example the computed discharges just upstream and downstream of the adjustable weir in sub-option 2a are depicted in Fig 5.1.

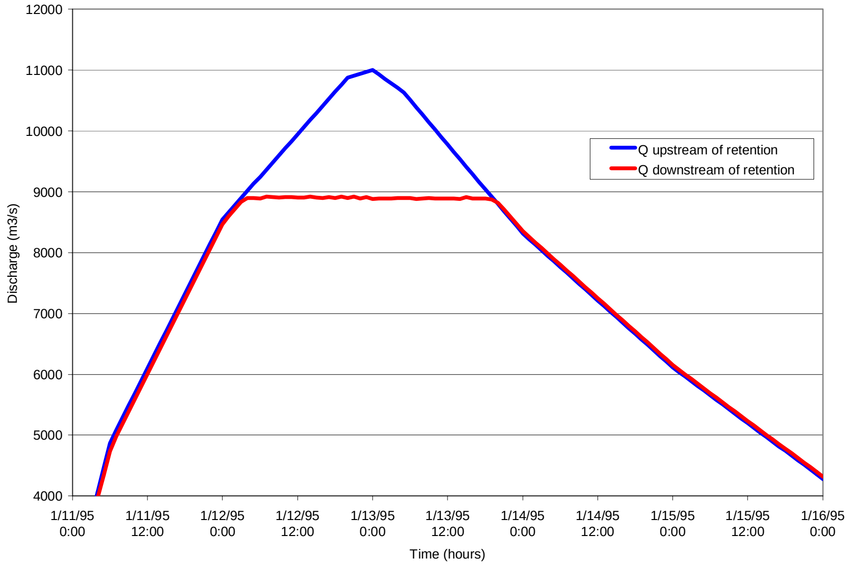

Fig 5.1 Case-study no 2, Sub-option 2a; Computed discharges just upstream and downstream of the adjustable weir.

5.2.3 Flood risk of Case study no 2

In designing the operation of the emergency retention polder uncertainties with respect to the strength of the dikes around both the Southern and Northern polder as well as the uncertainty in the magnitude of upstream discharges was not considered. In other words the operation options of the emergency retention polder were designed in a deterministic way. More precisely, it was taken care off that for an *assumed* maximum discharge of 10900 m^3/s (260 year return period), the storage capacity of the emergency retention polder is sufficient large enough for storing the volume of water contained in the 260 year flood-wave above the crest-level of the dikes surrounding the emergency retention polder. In addition it was ensured that the intake capacity of the weir(s) was sufficient large enough for allowing the timely storage of this volume of flood-water.
The possible flood-caused damage in the emergency retention polder of 77.9 M€ is negligible compared to the possible damage in the Southern and Northern polder (see Table 4.8). This was done in order to maximize the benefits in operating the emergency retention polder.

All flood risks of Case study no 2 are based on 3000 scenario's. In Table 5.5 and 5.6 these flood risks are arranged in accordance to the considered failure mechanism, viz:

- Considering the overtopping and piping failure mechanisms (see Table 5.5), and
- Considering the overtopping failure mechanism only (see Table 5.6).

For the corresponding flood-caused damages per scenario, reference is made to Appendix E.

From Tables 5.5 and 5.6 it can be concluded that not considering uncertainties in designing operation rules for an emergency retention reservoir is not justified, viz:

1. In case uncertainties are considered the emergency retention polder becomes <u>ineffective</u>. In other words *ineffective,* because the number of failures (see Table 5.5) compared to the reference situation without controlled flooding are the same for both the Southern polder (Option 1a: 39 ≅ 40 failures; and Option 2a: 41 ≅ 40 failures) and the Northern polder (Option 1a: 58 ≅ 57 failures, and Option 2a: 58 ≅ 57 failures). The fact that the number of flooding of the emergency retention polder compared to the reference situation reduces significantly is due to the fact that in Options 1a and 2a breaching of the dikes surrounding the emergency retention polders was not considered. In other words *ineffective*, because differences in flood risks are negligible compared to the reference situation, meaning for Option 1a: 62.8 M€ ≅ 63.7 M€; and for Option 2a: 65.0 M€ ≅ 63.7 M€ (see Table 5.5).
    a. One reason for this is the fact that the uncertainty in upstream discharge was not taken into account in designing the operation of the emergency retention polder. In some scenario's, upstream discharges were larger than 10900 m^3/s (see for instance Table E.2 in Appendix E). Hence larger discharges occurred than the assumed maximum discharge of 10900 m^3/s for which the operation of the emergency retention polder was designed, and
    b. Another reason is the fact that the strength of the dikes with respect to the overtopping and piping failure mechanisms was not taken into account in designing the operation of the emergency retention polder. In some scenario's the strength of dikes surrounding the Southern and Northern polders against overtopping and piping was less than for the 90 year (Q_{peak} = 9000 m^3/s) design discharge that was assumed in designing the operation of the emergency retention polder.
2. In case the piping failure mechanism (i.e. omitting one uncertainty) is not considered in the flood simulations, this results in the fact that the emergency retention polder *appears to become more effective*. In other words the number of failures (see Table 5.6) compared to the reference situation without controlled flooding reduce for both the Southern polder (Option 1b: 8 < 12 failures) and the Northern polder (Option 1b: 26 < 30 failures; and Option 2b: 28 < 30 failures). In addition differences in flood risks compared to the reference situation without controlled flooding become smaller respectively, Option 1b: 29.5 M€ < 39.4 M€; and Option 2b: 34.8 M€ < 39.4 M€; see Table 5.5). However, this concerns a <u>spurious relationship</u>, the emergency retention polder appears to become more effective, since in the design of the emergency retention polder uncertainties were not considered.

Further on from Case study no 2 it can be concluded that:

3. An emergency retention polder may limit the number of failures due to the overtopping failure mechanism but does not limit the number of failures due to the piping failure mechanism along the dikes surrounding the Southern and Northern polders. In addition in case the volume in the upstream flood wave that should be stored is much larger than the volume that can be stored in the emergency retention polder, this will result in an elongation of the exceedance of critical water depths for piping and this will even increase the flood risk due to piping, and
4. The Computational Framework allows for assessing the effect of the operation of an emergency retention polder on the flood risk in a catchment considering uncertainties in model parameters, such as the actual strength of a dike and the actual upstream discharge.

The fixed weir (options 1) appears to be more effective than the adjustable weir (options 2). This is due to the fact that for upstream discharges smaller than 10900 m^3/s, the discharges near the dikes along the Southern and Northern polders for a fixed weir are reduced to about 8550 m^3/s, while for an adjustable weir those discharges are reduced to 8900 m^3/s only. These higher discharge in case of an adjustable weir result more often in failure of dikes along the Southern and Northern polders and hence in a higher flood risk.

It is explicitly mentioned that the purpose of Case study no 2 is not to evaluate the usefulness of emergency polders that are presently considered in the Eastern part of the Netherlands. Case study no 2 is to be considered as a hypothetical but realistic case study from which valuable conclusions regarding the design of emergency retention polders can be drawn. The results clearly demonstrate that leaving out in the analysis uncertainties related to geotechnical failure mechanisms and upstream discharges, may yield incorrect indications of the flood risk and may, therefore, be misleading in the evaluation of measures to mitigate flood risk.

Table 5.5 Case study no 2, Number of Failures and Flood risk in 10^6 Euro on basis of 3000 runs (or scenario's), considering overtopping and piping failure mechanisms with model parameters as given in Table 4.4. Only flood risks considering the entire catchment are given.

Number of Failures (Overtopping and piping, 3000 scenario's)				
Simulation	**Entire catchment**			
	Southern polder	*Northern polder*	*Emergency retention polder*	*Entire Catchment*
Reference a	40	57	57	73
1a (fixed weir)	39	58	26	66
2a (adjustable weir)	41	58	27	66
Flood risk in 10^6 Euro (Overtopping and piping, 3000 scenario's)				
Simulation	**Entire catchment**			
	Southern polder	*Northern polder*	*Emergency retention polder*	*Entire Catchment*
Reference a	14.5	48.2	1.0	63.7
1a (fixed weir)	14.1	48.3	0.4	62.8
2a (adjustable weir)	16.7	47.8	0.5	65.0

Notes:
1. For Ref a, see also Table E.3 in Appendix E
2. For option 1a, see also Table E.5 in Appendix E
3. For option 2a, see also Table E.7 in Appendix E

Table 5.6 Case study no 2, Number of Failures and Flood risk in 10^6 Euro on basis of 3000 runs (or scenario's), considering only the overtopping failure mechanism and model parameters as given in Table 4.4, except for model parameters "Critical water depth for piping" and "Critical duration for exceedance of piping depth". Only flood risks considering the entire catchment are given.

Number of Failures (Overtopping only, 3000 scenario's)				
Simulation	**Entire catchment**			
	Southern polder	*Northern polder*	*Emergency retention polder*	*Entire Catchment*
Reference b	12	30	28	35
1b (fixed weir)	8	26	29	31
2b (adjustable weir)	12	28	27	31
Flood risk in 10^6 Euro (Overtopping only, 3000 scenario's)				
Simulation	**Entire catchment**			
	Southern polder	*Northern polder*	*Emergency retention polder*	*Entire Catchment*
Reference b	6.4	32.4	0.6	39.4
1b (fixed weir)	3.5	25.6	0.4	29.5
2b (adjustable weir)	7.1	27.2	0.5	34.8

Notes:
1. For Ref b, see also Table E.4 in Appendix E
2. For option 1b, see also Table E.6 in Appendix E
3. For option 2b, see also Table E.8 in Appendix E

5.3 Concluding remarks

Following concluding remarks can be made on basis of the results of both case studies, viz:

1. Case study no. 1 demonstrated that calculated flood risk reduces (i.e. positive effect) when system behaviour is taken into account. Please note that a case study focussing on demonstrating negative effects of system behaviour was not conducted,
2. Case study no. 1 demonstrated that for single and flood-wave dominated rivers yields that for same protection standards, downstream located polders benefit more from effects of system behaviour than upstream located polders,
3. Case study no 2 demonstrated that it is not correct to design safety-improvement measures in a deterministic way. Meaning that uncertainties (see section 2.3.3 and 2.3.4) in stochastic model parameters as well as in institutional uncertainties (for instance the decision to actually use an emergency retention polder) are to be considered jointly with the proposed safety-improvement measures,
4. Case study no 2 demonstrated that a deterministic analysis (no uncertainties considered) may yield incorrect indications of the flood risk, and may, therefore, be misleading in the evaluation of measures to mitigate flood risk,
5. For properly assessing the flood risk in a particular catchment a sufficient number of flood scenario's is to be considered (see section 6.4),

6. For properly assessing the flood risk in a particular catchment, it is essential to consider all relevant failure mechanisms jointly,
7. That part of the catchment area is to be considered, that is prone to effects of system behaviour and surrounding the area where a particular safety-improvement measure will be implemented. This is necessity for properly assessing the consequences of such safety-improvement measure on resulting flood risks. In addition the boundary conditions of the system should be independent for effects of system behaviour. If not these effects have to be accounted for in the applied boundary conditions,

Concluding it can be stated that using the Computational Framework, the effects of system behaviour on the flood risk in a catchment due to both passive and active interference by mankind, while jointly taking into account several failure mechanisms and various uncertainties can be assessed. This ability of the Computational Framework is a prerequisite for the proper assessment of flood risks.

6 Evaluation of Computational Framework

In this Chapter the more general aspects of the Computational Framework as applied in conducting Case study no 1 and Case study no 2 are discussed. In addition several suggestions for improvements are made. The general aspects discussed hereafter are:

- The hydraulic modelling,
- Modelling of failure mechanism,
- Breach development,
- The Monte Carlo analysis, and
- The assessment of flood-caused damage.

6.1 Evaluation of hydraulic modelling

In both case studies a relatively small catchment area and for Dutch standards a small safety-level of protections works (i.e. return period 90 years) was considered. The reason for doing so was to limit the number of scenario's as well as the computational time needed per scenario on a PC computer. Making the SOBEK-Rural/Urban suited to make runs on a cluster of PC's (parallel processing) would increase the computational efficiency. An other possibility would be to apply domain decomposition for those area's were less computational accuracy is needed, and hence computational efficiency could be improved. Both aspects are presently been considered at WL | Delft Hydraulics.

It is to be stated that the hydraulic system in both case studies was relatively simple with respect to the schematization and type of boundary conditions considered. For this reason based on the sampled values for the considered stochastic parameters (i.e. the actual values to be used in the various Monte Carlo runs, see sections 4.3 and 4.4), it was possible to determine beforehand whether a particular scenario (or Monte Carlo run) would result in flooding or not. For the scenario's not resulting in flooding, the corresponding flood-caused damage was taken as zero. Only the scenario's that might result in flooding were computed. In this way only a limited (i.e. about 90) number of the 3000 scenario's needed to be computed, resulting in a tremendous gain in required computational effort. It is to mentioned that for more complex river systems, the amount of Monte Carlo runs that can be excluded from computation beforehand might be much less. This will increase the required computational effort. With more complex river systems is referred to systems having a complex network of open channels, interrelated boundary conditions, boundary conditions of different type of nature (rainfall & tide & upstream discharges), regulated structures (storm-surge barriers, reservoirs) and so on. It is recommended to address this issue in future research activities.

The overtopping and piping failure mechanisms considered in both case studies could easily be implemented into SOBEK-Urban/Rural. In incorporating other as well an new generation failure mechanisms (see also section 6.2) no major difficulties are foreseen. This is due to the modular concept of the accommodated real-time control module.

For conducting the case studies a rather time-consuming and cumbersome method in preparing the required input for the flood scenario's was applied. For future practices, a special pre-processing module could be developed to make things more easier.

In the case studies, the river flow was modelled as one-dimensional flow and the flow in the polders was modelled as two-dimensional flow. This resulted in the fact that in the model schematisations of both case studies, it was necessary to define locations where exchange of one-dimensional river flow and two-dimensional polder flow was allowed. In case the river is also modelled two-dimensional, the flow automatically determines those dike-sections (i.e. 2D grid cells) for which yields that computed water levels in the river are higher than the local crest-level of the dike and in accordance the polder will be flooded. The same applies for possible flow from the polder towards the river. By defining failure mechanisms to all 2D grid cells (i.e. dike-sections), the computational results are not dependent anymore on the user defined locations where exchange of one-dimensional river flow and two-dimensional river flow is allowed. It is to be mentioned that the above can also be achieved by defining a connection between the river and each and every 2D grid cell located on the dike surrounding the polders. Except for being cumbersome, in case the number of such connection becomes to large, this will hamper again computational efficiency.

6.2 Evaluation of aspects related to Failure mechanisms

6.2.1 Evaluation of failure mechanisms as incorporated in the Case studies

In Case study no 2, analysis of the effectiveness of temporary storage in case of extreme river discharges, is based on comparing flood risks of the system with and without controlled inundation. For this a flood risk calculation concept is used based on a Monte Carlo analysis as explained in section 4.3.

In the system with controlled inundation, inlet of water into the designated storage area is initiated at some time, with the intention to reduce head levels in (parts of) the river system, in order to reduce the probabilities of occurrence of spontaneous dike breaches in the system. This process may be characterized as 'peak shaving' (see Fig 5.1). Yet, the possibility of spontaneous dike failure cannot be fully eliminated. The probabilities for some of the failure mechanisms may not even be significantly reduced, since they depend both on the duration of (high) river head levels as well as, or may be even more than, on the levels itself.

For example, stability of the inner slope of dikes depends greatly on phreatic pore pressures in the inner slope and/or pore pressures in soil layers below the dike. Pore pressure development tends to follow river head levels (of sufficiently long duration), with some delay, caused by phreatic storage capacity. Attenuation of flood waves for which yields that the volume to be stored is much larger than the actual storage capacity of the emergency retention polder, tend to elongate the duration of a river level in the river, therefore it might even increase the probability of slope failure.

The same applies to other types of geotechnical failure, e.g. piping, though to a lesser extent, and micro instability due to seepage erosion of the inner slope. However, probabilities of failure mechanisms associated with overflow and overtopping will be substantially reduced by peak shaving, at least as long as this causes sufficient reduction of river head levels. But again, if overflow or overtopping is not almost fully eliminated, even relatively small amounts of water running down the inner slope of the dike keep the process of intrusion of water into the dike going, building up pore pressures in the inner slope and, eventually, initiating slope failure and further erosion of the dike kernel.

It is therefore a realistic speculation that a methodology of comparison of flood risks of systems with and without retention which does not take into account (probabilities of) geotechnical failure mechanisms is awkward. Flood risks will be significantly affected by dike breach mechanisms which are not related to overflow or overtopping. Not taking these mechanisms into account may lead to over-estimation of flood risk reduction by temporary storage. This applies as well to spatial or structural measures to enlarge river discharge capacity.

Indications that this statement is valid follow from the results of the case studies, conducted in the course of this project. In these case studies only piping underneath the dikes has been included in the computation model, additional to overflow and overtopping. To this purpose a simple time dependent piping model has been implemented, which will be discussed into some detail in section 6.2.2 hereunder. Though this model is speculative to some extent and not validated, it was found to serve well the purpose of illustrating the previously described effects regarding the effectiveness of the retention system. The results of the flood risk case analyses have been discussed extensively in Chapter 5. Based on these results it may be clear that at least the most important geotechnical failure mechanisms should be included in the analysis in order to obtain realistic results for the evaluation of the effectiveness of temporary storage.

This raises a computational difficulty. Failure analysis models, for example for geotechnical (slope) failure of dikes or compound failure of hydraulic structures, is rather complex. Linking existing computation models for such failure analyses to the present computation model for flood risk evaluation would enormously increase the computational burden. Computational burden of the model as it is now, is already cumbersome. Consequently inclusion of geotechnical and structural failure in the flood risk analysis is only practically possible when simplified expressions for geotechnical and structural failure can be used, inevitably at the cost of loss of generality. Future research is being planned to rigorously improve computational efficiency of the flood risk analysis. If successful, this might re-enable the option of linking complex geotechnical and structural computation models.

A tractable option to include (probabilities of) geotechnical and structural failure in the Monte Carlo flood risk analysis in this report is the use of a response surface type approach. Basically, in such approach, a response surface, as a function of the (most) relevant random variables which play a role in some failure mechanisms, is constructed on the basis of a limited number failure mode calculations, varying each of the random variables one at a time. This way a set of response sample points can be obtained. The surface is constructed through fitting, based on the response sample points, thus providing an approximation to the "real" response surface. In case the sample points represent limit state points, the constructed surface represents an approximation of the limit state surface. The Monte Carlo analysis may then be carried out using the constructed limit state surface instead of the real one. This way the computational burden associated with the basic models for geotechnical or structural failure can be drastically reduced. For further details and literature review reference is made to [Waarts 2000]. The suggested simple approaches in the next section are inspired by this concept of constructed response surfaces, though they are less comprehensive.

6.2.2 Suggestions for improving failure applied failure mechanisms

The mechanism descriptions should address failure dependency of at least the most relevant load and strength parameters. Relevant load parameters include river head level or difference of potential head across the dike and time duration of high river levels. Strength parameters may be aggregated in critical head levels or critical time durations. Imaginably not all of the relevant uncertainties involved in geotechnical failure may be captured by failure mode expressions containing (critical) river head and (critical) time duration only. For this a generic type uncertainty term may be included mechanism expression.

The simplest case occurs when critical head level and critical time duration are mutually independent and can be characterized by one single value. The failure mechanism description may then be expressed as

Failure, associated with some specific mechanism at some location, occurs at time t when:

1. $h(t) \geq H_{crit}$, and
2. $(t - t_o) \geq T_{crit}$, and
3. outcome of some random experiment $re(P_{frs}) = 1$

where:

$h(t)$ = time dependent river head level,

H_{crit} = some critical head level, associated with the mechanism (initial failure); H_{crit} may be either a deterministic quantity, or the realization of a random variable,

t = actual time,

t_o = time point where h(t) initially exceeds H_{crit} ,

T_{crit} = some critical time interval, which may be either deterministic or the realization of a random variable, and

$re(P_{frs})$ = some random experiment with possible outcomes 1 or 0 and probability $P[re(P_{frs})=1]=P_{frs}$. P_{frs} is probability of failure of the residual strength.

The random experiment criterion may be helpful to specify the effects of components to the probability of failure which do not (evidently) relate to river head level or duration of discharge wave. Values or random characteristics of H_{crit} , T_{crit} and P_{frs} are mechanism type and location specific and should be estimated accordingly, preferably with help of probabilistic failure analysis.

A useful and straightforward extension of the model, allowing for mutual dependency of critical head level and critical time duration reads:

$$\{h(t)\geq H_{crit,1} \cap (t-t_{O,1})\geq T_{crit,1} \cap re(P_{frs,1})=1\} \cup \{h(t)\geq H_{crit,2} \cap (t-t_{o,2})\geq T_{crit,2} \cap re(P_{frs,2})=1\} \cup \ldots etc. \quad (6.1)$$

where $\cap$ and $\cup$ denote the event set operators *and* and *and/or*, and $H_{crit,1} < H_{crit,2} < \ldots etc$ and likewise $T_{crit,1} > T_{crit,2} \ldots$and $P_{frs,1} \leq P_{frs,2} \ldots$
In the sequel some of the relevant mechanisms for dike failure will be discussed in further detail.

(i) Dike failure due to overflow or overtopping
In Case study no 1 and Case study no 2, it has been assumed that dike breach starts, immediately following overflow or overtopping. In terms of the generic model, the parameters have been assessed to:

$$H_{crit} = H_{crest}\ ,\ T_{crit} = 0 \text{ and } P_{frs} = 1 \qquad (6.2)$$

This is a conservative approach, since, presumably, the inner slope of a dike has some capacity to withstand overtopping of water for some time. Basically two sequences of mechanisms, following overtopping, may yield final dike failure, namely:

- inner slope failure due to intrusion of overtopping water, building up pore pressures, followed by further erosion of the dike body until erosion of the crest.
- inner slope erosion of grass revetment and covering clay layer, followed by further erosion of the dike body until erosion of the crest.

For future flood risk analysis cases it might be considered to extend the failure mechanism *overflow or overtopping*, addressing the unmentioned additional sequences of mechanisms. The extended version of the simplified model seems to be indicated in order to distinguish between slope failure due to intrusion of overtopping water and erosion of the inner slope revetment. A first, not yet elaborated, approach may be:

$$\{h(t) \geq H_{crest} \cap (t-t_o) \geq T_{crit,sf} \cap re(P_{frs,sf})=1\ \} \cup \{h(t) \geq H_{crest} \cap (t-t_o) \geq T_{crit,er} \cap re(P_{frs,er})=1\} \qquad (6.3)$$

where $T_{crit,\ sf}$ denotes critical time to slope (sliding) failure, $T_{crit,\ er}$ the critical time to complete erosion of the inner slope revetment, and where, optionally, $P_{frs\ ,\ sf}$ and $P_{frs\ ,\ er}$ can be used to characterize uncertainty regarding final erosion of the dike kernel. We should be aware that present knowledge is still insufficient to objectively assess (statistical characterization of) these parameters. For the time being the use of tentative assumptions will be necessary, however, research regarding these topics is presently conducted as part of the Delft Cluster programme.

(ii) Dike failure due to piping
In the demonstration cases in this report the following model parameters have been adopted:

$$H_{crit} = H_{terrain} + \Delta H_{piping}\ ,\ \ T_{crit} = T_{piping} \text{ and } P_{frs} = 1 \qquad (6.4)$$

Where ΔH_{piping} and T_{piping} are assumed to be normally distributed random variables (see section 4.4 and Appendix C.2) Estimation of the statistics of these parameters is based in (quick and dirty) assessment of piping susceptibility (Bligh's rule) and tentative assessment of duration of the time from start of the internal erosion process until full development of piping.

For future flood risk analyses it is recommended to make parameter assessments for ΔH_{piping} based on location and situation dependent probabilistic piping analyses. Characterization of duration of the piping process is difficult. Models which describe time dependency of the internal erosion process are neither available nor under development. Crude indications can only be based on some observations of model tests and even more tentative guesses, based on actual failure cases. According to expert opinions, there is little evidence to assume that residual water retaining capacity of a dike is significant after piping (= formation of erosion canals throughout the base of the dike). Thus, there are no arguments to significantly adjust P_{frs} from this point of view. However, as far as P_{frs} reflects also uncertainty regarding initial breach of a cohesive cover layer near the inner toe of the dike (which, if such layer is present, is a necessary condition for piping), it may need adjustment, based on a probabilistic analysis.

(iii) Dike failure due to inner slope failure (not due to overflow or overtopping)

This potential failure mechanism has not been incorporated in Case study no 1 nor in Case study no 2. Yet, it may be one of the most relevant failure modes in the evaluation of the effectiveness of flood risk mitigation by temporary storage. The (probabilistic) analysis is somewhat more complex, compared with piping analysis, and framing the results of such analysis into the (extended) simple expressions presented in this section may be more cumbersome, but not prohibitive as far as can be seen now.

Basically failure of the inner slope depends on pore pressure development within and below the dike in response to head level development in the river (both actual extreme level as well as duration). Uncertainties of pore pressures relate to uncertainties of the geohydrological characteristics and parameters, as well as to the initial saturation level of the dike body, as a result of weather conditions prior to the extreme river discharge event. Both uncertainties regarding pore pressures as well as uncertainties of shear strength properties play a role in the probability of inner slope failure. Whether or not slope failure implies dike failure depends on residual capacity to retain water after slope failure.

For the simplified expression of dike failure due to inner slope failure the use of a series of conditions seems indicated:

$$\{h(t)\geq H_{crit,1} \cap (t-t_{O,1})\geq T_{crit,1} \cap re(P_{frs,1})=1\} \cup \{h(t)\geq H_{crit,2} \cap (t-t_{o,2})\geq T_{crit,2} \cap re(P_{frs,2})=1\} \cup \ldots etc. \quad (6.5)$$

where $H_{crit,1} < H_{crit,2} < \ldots etc$ and likewise $T_{crit,1} > T_{crit,2}$ .Assessment of the $H_{crit,i}$ and $T_{crit,i}$ may be based on actual probabilistic slope failure analyses using real or assumed actual conditions regarding (local) geohydrological characteristics and parameters and geomechanical parameters.

Estimates of $P_{frs,i}$ may reflect failure of residual strength after initial slope failure. These estimates can be obtained from a presently conducted research project in the Delft Cluster program (Failure Mechanisms and Residual Strength).

(iv) Simplified models for structural failure

Similar to the expressions for geotechnical failure of dikes, simplified models for failure of water retaining hard structures may be developed. However such models have not been elaborated in the Case study no 1 and Case study no 2.

6.2.3 Correlations and mutual interference of failure modes; unresolved questions

(i) Correlation among different failure modes

Mutual correlation among the various different failure modes is evidently present. Many of the potential failure modes (both geotechnical as well as structural) will be correlated through dependency on exerted load, i.e. river head levels and, to a lesser extent, on the shape of the river discharge wave. For this part no specific precautions must be taken regarding the formulation of simplified failure modes. Mutual correlation among failure modes is automatically accounted for in the Monte Carlo simulation procedure, as the evaluations of different failure modes utilize the same realizations of river head levels and river head level durations.

As to correlation, induced by (common or strongly correlated) soil parameters, the matter might be somewhat more complex. Different geotechnical failure modes may imaginably be affected by the same, or at least strongly related, soil properties. For example, probabilities of piping and inner slope failure are mutually dependent in situations where these mechanisms are initiated by (uncertain) uplift of clay or peat cover layers near the inner toe of the dike, i.e. when the pore pressure in deep sand layer exceeds dead weight of the overlaying clay/peat stratum. Taking into account such dependency requires either that the weight of the clay/peat cover layer is explicitly included in the formulation of simplified failure modes, or that in one way or another correlation among these failure modes can be taken into consideration in the Monte Carlo analysis set up. Tractable options for the latter are still to be elaborated.

(ii) Correlation in space and time

Load or load effect related variables in the analysis of probability of dike failure are mainly derived from river discharge statistics. River discharge statistics reflect the random character of annual extremes of river discharge. Key feature of load or load effect related variables is strong correlation at a regional scale, strong correlation at time scale during an annual extreme river discharge event and lack of correlation among subsequent annual events. Strength of the structure as a whole, or component strength of structure components, show more or less the opposite. Namely, rapid decaying correlation in a spatial sense, but relatively strong correlation in time over the sequence of annual extremes. For example, fundamental soil properties, though in itself uncertain, do not, or at least are considered not to, alter in time. And as far as changes in time do take place, the process of development will be rather deterministic instead of purely stochastic. For the setup of the Monte Carlo simulation procedure, as outlined in section 4.3, this has no specific implication, since it focuses on quantification of the annual flood risk, i.e. the flood risk associated with one single (yearly) extreme discharge event. Strong correlation in time of strength related variables may play a significant role when considering a sequence of such year events, i.e. when quantifying the flood risk during some reference time period of, say, 10 or 50 years. Ignoring correlation in time in such analysis would yield over estimation of flood risk.

(iii) Mutual interference of failure modes

Occurrence of dike or structural failure of the water retaining system at any specific time point and at any (designated or free) location is basically a compound of different potential failure events, dominated by *and/or* combinations. Suppose that at some specific time point and at some specific location two or more different failure modes tend to become active.

Then, the order of computational evaluation will not affect the occurrence of dike failure nor will it, assuming that any active failure mode initiates the same process of breach growth, affect the evaluation of flood risk. But, since the cause of dike failure will be attributed to first mechanism in the computational analysis, it may affect to some (minor) extent the scores of relative contribution to dike failure attributed to the various mechanisms involved.

A similar computational phenomena occurs when at some time point dike breach (and growth of the breach) is initiated at different locations. Basically, initiation of breach at one location may stop the initiation of breach at the same time at other locations. However, whether it does and, if so, at which one of the potential locations breach indeed occurs, may depend on small physical details, which may not be accounted for in the Monte Carlo simulation. In the simulation analysis breach will occur at all the locations, and it is questionable whether sufficiently justified criteria, if at all, can be found to stop breach growth at those locations where the conditions regarding failure tend to be mitigated by initial breaching elsewhere (see also section 6.3). However, this may significantly affect the development of inundation pattern and consequently the evaluation of flood risk.

6.3 Evaluation of breach development

Both in Case study no 1 and Case study no 2 a pre-defined breach-growth-scenario (i.e. final breach width, see Table 4.4) was defined. Meaning that in case an initial breach is initiated by a particular failure mechanism, irrespective of the hydraulic conditions this breach will always grow until its final breach width is attained. Recently a new breach growth formulation (i.e. Verheij-vdKnaap (2002), see Appendix B.2) was implemented into SOBEK-Rural/Urban, that computes the breach growth of an initial breach as function of dike-strength parameters and the actual flow conditions in the breach. Using the Verheij-vdKnaap (2002) formula, means that breach growth stops as soon as the flow velocity in the initial breach becomes smaller than a user-defined critical shear stress velocity. The Verheij-vdKnaap (2002) formula is to be considered as a step-forward towards improved formulations for breach growth. But, nevertheless it is anticipated that future research might lead to even more improved breach growth formulations.

6.4 Evaluation of Monte Carlo analysis

The Monte Carlo analysis is evaluated for Case study 1 option 1 only.

6.4.1 Number of Monte Carlo simulations

One of the basic questions when performing a Risk Analysis using Monte Carlo is the question with respect to the accuracy: is the previous result based on 3000 runs accurate enough or not? Let us try to answer that question. First of all we should make a distinction between the uncertainty in the number of runs in which inundation occurred and the uncertainty in the consequences given inundation. Let us first consider the uncertainty in the number of runs leading to inundation.

(i) Uncertainty in number of runs in which flooding occurs:

In general, when performing a Monte Carlo simulation with failure probability p, the expected number of failures and the standard deviation in a simulation with n runs are given by, respectively:

$$\mu = pN \tag{6.6}$$

$$s=\sigma = \sqrt{(pN)} \tag{6.7}$$

So the coefficient of variation is equal to :

$$V = \sigma/\mu = 1/\sqrt{(pN)} \tag{6.8}$$

Due to the Central Limit Theorem, the distribution of the number of failures has a normal distribution for large values of N.

According to a standard FORM analysis for the present case (no details presented), the overtopping probability for each of the four individual critical sections is equal to 0.012. The probability of overtopping somewhere along the two dike-rings is equal to 0.016. As a result, the number of runs having one or more failures to be expected in a Monte Carlo simulation is equal to pN = 3000 * 0.016 = 48. The variation here is $\sigma = \sqrt{pN} = \sqrt{48} = 7$, i.e. V = 14 %. The observed amount of runs with failure should not deviate too much from the expected range (say for instance $\mu \pm 2\sigma$). In this case study, the expected range is between 34 and 62. Therefore, it may be concluded that the present Monte Carlo analysis with 60 runs having failure out of in total 3000 runs is okay. Note that if it is not okay we have the opportunity to make a correction as the exact values is already known in advance. In this case the correct answer comes from an ad hoc FORM calculation. When applying the procedure to dike-rings in the Netherlands the computer code PC-Ring or any other reliability calculating program would provide this information.

(ii) Uncertainty in the consequences given inundation occurs:

Let us now consider the scatter in the resulting damage, given that inundation has occurred. In fact, the only interesting issue is the estimation of the conditional mean damage given inundation occurs. If we accept an error of 10 percent in this estimate, a sufficient number of runs obtained if:

$$\sigma(D|F) /\sqrt{(N)} < 0.10 *\mu(D|F) \tag{6.9}$$

Here N is the number of runs leading to inundation and $\sigma(D|F)$ and $\mu(D|F)$ are the estimators of the conditional mean and standard deviation following from the Monte Carlo sampling. The choice of 10 percent accuracy for the estimate of the average damage, of course, is fully arbitrary. Let us apply the above to Case Study 1 option 1. The damage D in the runs where an inundation F occurred (60 in total), has a mean value $\mu(D|F)$ = 3.5 billion Euro and a standard deviation $\sigma(D|F)$ = 2.3 billion Euro. So the coefficient of variation in the consequences of flooding is large: V(D|F) =66 %.
The uncertainty in the expectation for the 60 observations, is equal to $2.3 / \sqrt{60} = 0.3$ million Euro, corresponding to a coefficient of variation of 9%, which is less than the 10 percent

used in Equation (6.9). We can conclude that enough Monte Carlo simulation have been carried out.
Please note, however, that in fact what we need here is a preposterior Bayesian analysis where the costs of extra Monte Carlo runs (or scenario's) and their improvement in the final computed flood risk are weighted against each other. In Appendix F an example of such preposterior Bayesian is given, followed by a theoretical explanation on how to apply this preposterior Bayesian analysis to Monte Carlo simulations.

(iii) Combined uncertainties due to number of runs and flooding consequences:

Note that the total variation in the risk R (assuming independence of the probability of failure and the probability of damage) is equal to:

$$V(R) = \sqrt{(0.14^2 + 0.09^2)} = 17\ \% \qquad (6.10)$$

Concluding:
As discussed before, one should keep in mind that this Monte Carlo simulation is not meant to calculate the failure probability. The failure probability has been already calculated using FORM in an earlier calculation. Corrections for possible errors in the estimate of the failure probability can directly be corrected. This is not so relevant in Case study no 1, where the failure probability is in the order of 0.01 (i.e. 90 years return period). It may, however, be more important for cases with much lower failure probabilities. If we consider only the uncertainty in the consequences the value of 10% seems to be quite sufficient for the purpose of the analysis.

6.4.2 Checking for outliers in stochastic variables

In Case study no 1; entire catchment, 3000 scenario's, both overtopping and piping failure mechanism, it occurred that in the first 1000 scenario's an extremely large discharge scaling factor was sampled, that increased the flood risk based on these 1000 scenario's considerably (see Fig 6.1). However, after 3000 scenario's a reliable flood risk estimation was attained. In Fig 6.2 the damage for each of these 3000 scenario's is given. Looking more into detail we see that the expected maximum value for the discharge factor (the most important random variable) is 1.7, corresponding to the 1/3000 fractile of the used distribution (or in other words in 3000 samples the value of 1.7 should be exceeded only once). A value of 2.1, corresponding to a 1/22000 fractile has been obtained in the simulation, leading to a peak in the economic damages (15 billion Euro). Such a value should occur only once in every 22000 Monte Carlo runs. It can be concluded that the set of 3000 runs that has been obtained in the present simulation represents a more or less extreme case. One might consider to make a correction of this high value, however, it would have a minor influence on the final result only.

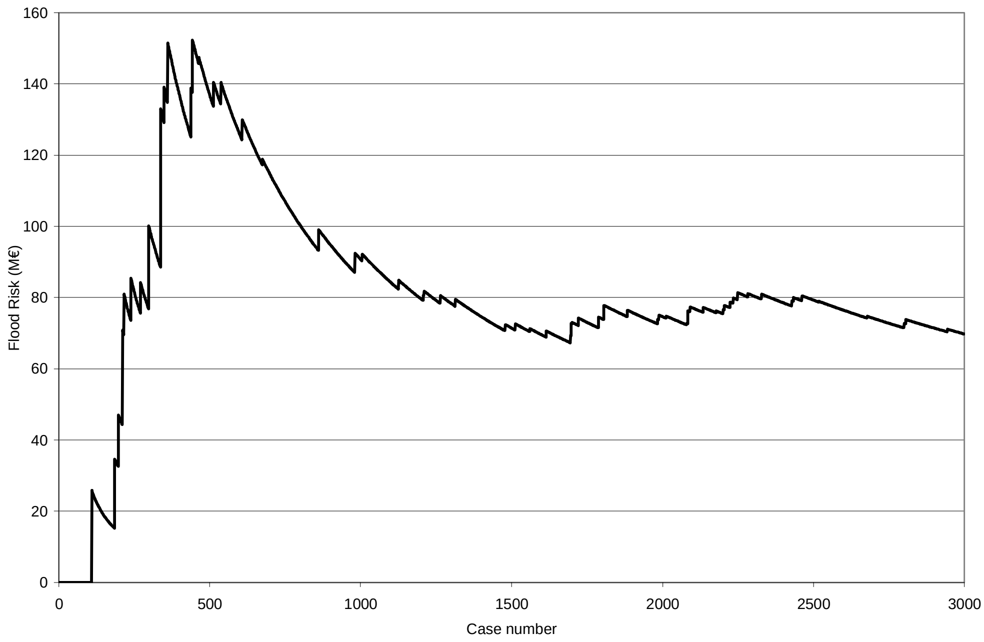

Figure 6.1. Case study no 1; Entire catchment, 3000 scenario's, both overtopping and piping failure mechanism, Flood risk as function of number of conducted scenario's.

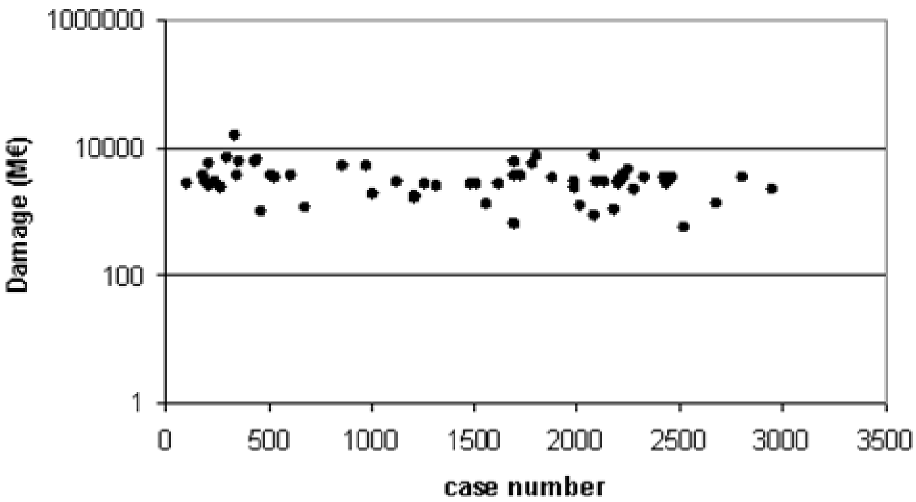

Figure 6.2. Case study no 1; Entire catchment, 3000 scenario's, both overtopping and piping failure mechanism, Flood-caused damage for each considered scenario.

6.5 Evaluation of Flood-caused damage assessment

Resulting flood-caused damages in different scenario's are very similar. The reason for this is that in general the inundation depths are falling in the largest maximum water depth class, applied in damage assessment (see Fig 4.5, in section 4.5). The maximum water depth class for residential areas is 5 metre and for all other type of land-use the maximum water depth class in only 3 metres

As mentioned earlier the possibility of causalities was not considered, since it was decided that there should be ample time for the evacuation of local inhabitants.

Further on the consequences of the spread of contaminated silt and toxic agents was not considered. At the time of conducting both case studies, the two-dimensional water quality model enabling this type of simulations was not yet available. In addition no tool for assessing damage due to contaminated silt and toxic agents is presently available.

Furthermore, other types of damage are neglected in this study, such as the indirect economic effects, damages to environmental and cultural values, and the societal impact. For a further analysis of these different type of damages, reference is made to the report of the Delft Cluster project DC 02.03.03 “Consequences of floods”.

7 Main Conclusions and Recommendations

7.1 Brief description of project activities

In this study a concept of flood risk evaluation has been applied to quantify effects of river system behaviour in a regional setting. The concept (or Conceptual Framework) is based on an integral catchment-wide flood risk analysis; the overall product of probability of flooding and their consequences. The Conceptual Framework consists of an Institutional Framework and a Computational Framework. Although several institutional aspects have been briefly outlined, the study has mainly focussed on the development of the Conceptual Framework.

With effects of river behaviour is meant that the safety of a particular dike-ring might depend on the safety of other dike-rings. These effects might be positive or negative. Positive effects of system behaviour occur in case the hydraulic load on a dike-ring is reduced due to the failure of an upstream located dike-ring. Negative effects of system behaviour occur in case due to a local dike breach, flood water from a major river branch flows into a minor river branch (f.i. from Waal to Meuse). The driving mechanism in system behaviour is, of course, the mutual interaction between geo-technical failures of dikes and the hydrodynamic response of the river system to it.

Using a simplified version of the Computational Framework two hypothetical but realistic case studies were conducted. The first case study referred to a system without any human interference. In the second case study the operation of an emergency retention polder aiming at mitigating the downstream flood risk was taken into account. This Computational Framework consisted of: hydraulic modelling of river system behaviour (i.e. one-dimensional river flow and two-dimensional polder (or dike-ring) flow; modelling of geo-technical failure mechanisms (i.e. overtopping and piping); flood risk analysis (i.e. Monte Carlo analysis) and flood-caused damage assessment (i.e. Dutch Standard Damage model, HIS-SSM, considering economic damage only).

7.2 Conclusions

The study resulted in a good insight into the requirements of the Computational Framework. Requirements that concern the incorporation of hydraulic aspects, geotechnical and structural aspects, flood risk aspects, and uncertainties in physical processes as well as in societal and institutional aspects.

Due to practical limitations and the current state of required technology/knowledge, simplifications in the Computational Framework applied for conducting the two case studies were made. Nevertheless, a significant step towards a more sophisticated Computational Framework was made.

Case study no 1 demonstrated that calculated flood risk reduces (i.e. positive effect) when system behaviour is taken into account. Further on that for single and flood-wave dominated

rivers having same protection standards yields that downstream located polders benefit more from effects of system behaviour than upstream located polders. Please note that a case study focussing on demonstrating negative effects of system behaviour was not conducted.
Case study no 2 demonstrated that a deterministic approach for designing an emergency retention polder, meaning that uncertainties in geo-technical failure mechanism, magnitude of upstream discharges and so on are not taken into account, may yield incorrect indications of the flood risk, and may, therefore, be misleading in the evaluation of measures to mitigate flood risk.

More general conclusions from conducting both case studies are:

1. The Conceptual Framework was generic enough for conducting both case studies. Meaning that the uncertainties to be considered in geo-technical failure mechanisms and boundary conditions could easily be implemented. The same yielded for the operation rules considered for the emergency retention polder,
2. For properly assessing the flood risk in a particular catchment a sufficient number of flood scenario's (or Monte Carlo runs) is to be made,
3. That part of the catchment area is to be considered, that is prone to effects of system behaviour and surrounding the area where a particular safety-improvement measure will be implemented. This is necessity for properly assessing the consequences of such safety-improvement measure on resulting flood risks. In addition the boundary conditions of the system should be independent for effects of system behaviour. If not these effects have to be accounted for in the applied boundary conditions,
4. For properly assessing the flood risk in a particular catchment, it is essential to consider all relevant failure mechanisms as well as all proposed safety-improvement measures jointly.

From statistical analysis on the case study (more precisely: Case study no 1, option 1) results, it was shown that the applied Monte Carlo analysis provides sufficient accuracy in computed flood risk. A characteristic feature of flood damage in our calculation appeared to be that it depends significantly on the circumstances and conditions which lead to dike breach (see fig. 6.3). That is, in the calculation the flood damage may be seriously affected by the actual values of the stochastic parameters. This is particularly true for flood damage originating from simultaneous dike breaches at different locations (in the same or in different areas) where inflow of river water through the breaches is affected by the river system behaviour. The adopted Monte Carlo approach takes this into account.

Concluding:
For determining proper flood risks, it is a *prerequisite* that effects of system behaviour (both passive and active interference by mankind), geo-technical failure mechanisms, uncertainties and safety-improvement are jointly assessed. The Computational Framework provides this required functionality.

7.3 Recommendations

Current Dutch practice is to assess flood-safety on basis of single dike-sections or a complete dike-ring. Mutually interactions between various dike-rings are not yet considered. Taking the conclusions given above into account, it is strongly recommended to determine in future flood-risk in line with the concept of the explained Computational Framework, that allows for taking into account effects of system behaviour on flood risk. This will introduce the need of a model combining the hydraulic modelling of river system behaviour, geo-technical failure mechanisms, uncertainties in various model parameters, as well as operational options of various structures accommodated in the river system. Using such model the consequences of a particular safety-improvement for the entire catchment area can be assessed. Such model should be maintained and constantly be updated for newly implemented safety-improvement measures.

It is to be mentioned that the Computational Framework applied for conducting both case studies still is subject to further improvement. In Chapter 5 and 6 several recommendations for further improvement are discussed. In short these recommendations are:

Related to hydraulic modelling:

1. To improve the computational efficiency by introducing parallel processing and domain decomposition into SOBEK-Rural/Urban,
2. To develop a post-and pre-processing tool for reducing the effort in making and analysing the several hydraulic computations,
3. To model the river as two dimensional flow in order to leave to the system to find the most appropriate location for dike-failure or overtopping,
4. To develop an algorithm capable of determining beforehand for complex river systems such as the Dutch river delta for which scenario’s no flooding is to be anticipated. Hence making it possible to reduce the number of scenario’s to be computed,

Related to Geotechnical and structural aspects:

5. To include additional geo-technical failure mechanisms, such as for instance dike failure due to inner slope failure,
6. To include failure mechanisms for water-retaining hard structures,
7. To incorporate advanced formulations for geo-technical failure mechanisms, that account for the residual strength of dikes after an initial breach has occurred,
8. To incorporate improved descriptions for the development of a breach in a dike as function of actual occurring hydraulic loads,

Related to determining flooding probability:

9. In the case studies considered the failure probabilities are so high that a straightforward (Crude) Monte Carlo analysis was performed. However, when in future more complex cases will be investigated, it might be worthwhile to adopt Importance Sampling Monte Carlo analysis in order to reduce the number of required Monte Carlo simulations,
10. In order to attain at reliable flood risk for a reduced number of Monte Carlo simulations, it is worthwhile to check for outliers (i.e. extreme sampled values) in stochastic input variables,

11. To include a preposterior Bayesian Analysis for evaluating the Monte Carlo analysis applied in determining flood risk, and

Related to determining flood-caused damage:

12. To include water-quality aspects (spread of contaminated silt and toxic agents), indirect economic benefits, damage to environmental and cultural values, and societal aspects in the evaluation of flood risks.

Both in policy making and during crisis situations the concept of system behaviour will have influence on decision making. In this study a very limited analysis of institutional aspects has been carried out. A further investigation of these aspects is, therefore, recommended.

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APPENDICES

A Summary of relevant Studies and Literature related to the assessment of Effects of River System Behaviour

A.1 Jager, F.G.J. (1998)

In assignment of Dienst Weg- en Waterbouwkunde (DWW), Rijkswaterstaat, research on the effects of system behaviour for a single river system was conducted by F.G.J. de Jager during his MSc thesis work at the Twente University of Technology. More precisely, the effects of system behaviour considering following dike-rings, viz: Betuwe /Tieler- en Culemborgerwaarden (nr. 43); Land van Maas en Waal (nr. 41); Bommelerwaard (nr.38); Land van Altena (nr. 24); and the Alblasserwaard en de Vijfheerenlanden (nr. 16). All these dike-rings are assumed to flooded by the river Waal only, while most of them are adjacent to other main Dutch rivers as well. In the research, the emphasis was on determining the consequences of applying a safety approach considering only one dike-ring and a safety approach considering a system of dike-rings. The research clearly demonstrated the positive effect of system behaviour on flood risk.

A.2 Picaso (2000-2001)

In previous reliability and risk calculation projects in The Netherlands (TAW, Marsroute, Picaso, VNK, etc.) only isolated dike-rings were considered. However, even a single dike-ring should be considered as a system. Usually, in the calculation of the inundation probability for an individual dike-ring, the failure probabilities for each failure mechanism and for each section are evaluated separately. These failure probabilities are then summed up to get the system reliability. When summing up the correlation between the various dike sections and the various mechanisms has to be taken into account. In terms of reliability this implies carrying out a system analysis.

In a risk analysis for a dike-ring extra considerations, compared to the reliability analysis, must be taken into account. The point is that the consequences of flooding may depend on the number of dike sections that fail, on the order in which they fail and on their physical interactions. In PICASO this has been done in a simplified manner: it has been assumed that only the weakest dike section along a river branch fails during a flood, while all others are released; the dike sections along other river branches are assumed to be unaffected.

A.3 Tillie, J. (2001)

The research carried out by Judith Tillie is to be viewed in the context of the possible use of emergency retention areas (noodoverloopgebieden, Commission Luteijn) in the Netherlands. The aim of her MSc Thesis work was the design of a methodology for making decisions on the use of emergency retention areas in times of large floods on the river Rhine. Using the 1D SOBEK Rijntakken model, the propagation of different flood waves (i.e. Q_{max} at Lobith of respectively 16000, 17000, 18000, 19000 and 20000 m^3/s was considered, where the Q= 16000 m^3/s flood-wave is referred to as the standard flood-wave. For each flood-wave, different scenario's related to the combination of various emergency retention areas were made. On basis of a multi-criteria analysis, the most promising scenario's were selected. It appeared that a flood-wave having a different shape and peak-discharge could be transformed to a standard flood-wave shape, having a peak-discharge accounting for the volume of water contained above a discharge level of 16000 m^3/s. The decision methodology was developed on basis of one downstream water level. It is advised to take in decision making also into account maximum water levels at several upstream locations along the Rhine river. In the research adjustable weirs were used for diverting flood-water towards the emergency retention areas. It is advised to further investigate if adjustable weirs are an realistic option.

A.4 Vermeij, M. (2001)

The research carried out by M. Vermeij is also to be viewed in the context of the possible use of emergency retention areas (noodoverloopgebieden, Commission Luteijn) in the Netherlands aiming at improving flood risk. The research focussed on three aspects regarding the use of emergency retention areas, viz: a) How to determine the dimensions of an intake weir; b) What are the advantages and disadvantages of possible alternative dimensions of such weir; and c) How large is the effect of the weir on the water level draw-down along the main river.

In the research a model was developed using the DUFLOW software package was developed. Using this model, the required dimension of a weir for achieving a particular water level draw-down on the main river was established. From a sensitivity analysis it appeared that the required length of the weir-crest is very sensitive to variations in the peak and shape of the flood-wave, the bathymetry and roughness of the river, as well as to the dimensions of the emergency retention area. It is advised to further investigate the application of adjustable weirs.

A.5 Mao-Ming Zhou (1995)

Hao-Ming Zhou, Towards an operational risk assessment in flood alleviation, Delft University Press, 1995.

The research aimed at supporting the decision making process in establishing the optimal safety-level of polders against flooding by means of embankment heightening. The research focussed on that part of the Dutch river system, where the water movement is governed by both tidal waves and flood waves. Hereafter called the sea-river dominated area. A framework was developed for determining the optimal dike level, while taking into account dynamic aspects, multiple objectives and uncertainties. The problem was divided into three steps, viz: evaluation of the flooding process, evaluation of damage, and determination of the uncertainty. The framework was applied to four polders in the Netherlands, located in the sea-river dominated area. The main conclusion is the fact that a decision for dike heightening is based on comparing large but uncertain benefits against relative small and certain cost. Except for this, the uncertainty whether or not an initial breach will develop plays an import role in the decision making process.
The importance of system behaviour on flood risks is often referred to in the PhD thesis work. Quote page 68 "A change in flood protection (dike heightening) for one polder will

therefore not only change the flooding for the particular polder but also for the other polders. The different alternatives for flood protection are interdependent. This strongly increases the dimensionality of the decision problem as many combinations have to be considered” unquote.
From a case study it was concluded that the heightening of dikes around one polder had a small effect on anticipated damage in another area. Further on it was remarked that the phasing and implementation of dike heightening is of importance due to effects of system behaviour. This effect was not further elaborated in the case study.

A.6 IRMA Living with Floods (2001)

IRMA Living with floods (resilience strategies for flood risk management and multiple land-use in the lower Rhine River basin), Executive Summary, Alterra, IHE, RIZA, TUD, WL|Delft Hydraulics, November 2001. Sponsored by IRMA (EU) in the context of the IRMA-Sponge research programme and sponsored by Delft Cluster.

Flood risk management in the lower Rhine River basin (downstream from Cologne) relies on flood control by dikes for many centuries. This has resulted in an ever increasing sense of safety and, subsequently, in increased investments in the protected areas. In the long term, however, this causes an increase in vulnerability to flooding and a recurrent call to further control the floods, with many negative impacts on natural and cultural landscape values, and eventually also on society at large.
The objective of the project was to design and evaluate alternative flood risk management strategies which are applicable for the long-term (50-100 years) and better take into account the uncertainties that are inherent to lowland rivers. Two different strategies were elaborated, based on the principle of resilience and living with floods: 1) ‘compartmentalisation for detention’ and 2) ‘green rivers for discharge’. It was found that these alternative strategies have many advantages from a sustainability point of view, but are difficult to implement. They require huge investments and have enormous impact on local and regional scales, whereas the advantages are obvious primarily from a long-term point-of-view and in a larger spatial-scale frame.

A.7 VNK/FLORIS (2003)

The Flood Risk (FLORIS) project has been initiated to assess the safety of all flood prone areas in the Netherlands. FLORIS is divided into the following four subprojects:

1. Assessment of the probability of flooding of dikes and dunes
2. Assessment of the probability of flooding of hydraulic structures
3. Determination of the impacts of floods
4. Development of methods to take into account uncertainties in subprojects one to three

With the knowledge developed in these subprojects insight will be gained in the current flooding probabilities, the impacts of flooding, and by combining these elements, flood risks. In the current methodology for assessment of flooding probabilities every dike-ring is considered separately. Effects of system behaviour are not taken into account in these calculations. Flooding probabilities calculated in this FLORIS project can therefore generally be considered as upper limits. However, in some occasions system behaviour can result in higher discharges. For example when a flood in the Rhine river system creates a shortcut to the IJssel or Meuse river, and thus results in an increase of the discharge in these systems.

Two examples of the inclusion of system behaviour in the results of FLORIS are given below. Firstly it is currently considered to determine the probability of flooding for secondary dikes. These are dikes which are only loaded by hydraulic conditions when a flood occurs due to a breach in a primary dike. Secondly, in the hydraulic boundary conditions used for the calculation of failure probabilities, also the interaction is considered between the river discharge and the closure of the storm surge barrier.

A.8 Room / space for rivers program

In order to be prepared for future developments, such as climate change, several projects are carried out to study the possible solutions to cope with increasing discharges in the river system.

Within the Room / space for rivers program it is investigated how increases in the discharge capacities of the Rhine and Meuse river can be achieved with measures such as dike relocation and the deepening of the flood plain.

A.9 Spankracht study

Within the “Spankracht” project it is studied which measures can be taken to deal with expected long term changes in the river system. A design discharge of 18000 m^3/s in the Rhine river is assumed to be possible in the future. Strategic solutions measures such as retention, adjustments to the discharge distribution and lowering of the flood plains have been proposed.

A.10 Governmental Committee Emergency Retention Areas, Commission Luteijn (2001-2002)

A special committee (i.e. Commission Luteijn) has been installed to advise the government on the possibilities of the application of emergency retention areas (noodoverloopgebieden). The committee has proposed some areas which could be suitable for emergency retention and further studies are being undertaken to study the feasibility of the proposed concept. It has to be noted that this type of solution includes many uncertainties, as is noted by Vrijling [2002].

Also other types of research are to some extent linked to system behaviour, for example studies undertaken on the impacts of floods (in relation to risk calculations), and more general advices which propose flood defence strategies for a whole catchment area.

B Summary of relevant Studies and Literature related to Dike breach development

B.1 Visser, P.J. (1998)

Breach Growth in sand-dikes (PhD Thesis), Delft University of Technology, 1998.

In the PhD Thesis a mathematical model is formulated for predicting the growth of an initial breach in a dike and the discharge flowing through this breach. It is assumed that the dike consists of sand and that revetments provide no resistance to the erosive process. In the development of the initial breach five phases are discerned. In the first two phases the breach erodes a path through the dike, while the magnitude of the discharge hardly increases. In the third phase the dimensions of the breach as well as the discharge through the breach increase rapidly. In the fourth phase the dike has collapsed and the breach grows mainly in width only, while the flow through the breach remains super-critical. In the fifth phase the flow through the dike-breach has become sub-critical and as a result the breach growth speed diminishes. In case the flow velocities in the breach become smaller than the critical Shield value, the breach will not further develop. The breach growth process in the last two phases depends on the sub-soil of the dike, the revetment on the water-side of the dike, and the elevation of the area in front of the dike. An essential aspect in the breach growth model are the sediment formula's describing the picking-up and transport of dike material. The mathematical model including the selected sediment formula's (f.i. Bagnold-Visser for the first three phases) showed a good fit with both the Zwin'94 prototype experiment as well as with laboratory experiments (Caan, 1977).

B.2 Verheij, H.J. (2002)

Verheij, H.J.: Modification breach growth model in HIS-OM; WL|Delft hydraulics, Q3299, November 2002, Delft (in Dutch).

In the framework of a research project funded by the Ministry of Public Works in the Netherlands, the time-dependent grow of the breach width in cohesive embankments has been studied. The width of a breach is important because it determines the amount of water flowing into a polder (together with the water levels and the duration of the flood). Clearly, the breach width is an important parameter in inundation simulations.
Up to now, validated breach growth formulas were only available for sand dikes. The fundamental research resulted in an analytical formula based on the physical behaviour of soils as a result of water flowing over the soil and water inside the soil (in the pores and the soil itself). Water content, soil structure, sizes of soil particles, strength of the soil, and pore pressure gradients are important parameters. In principle, the resulting physical-based formula enables the prediction of the breach growth in a dike due to the inundation of a polder. However, due to lack of time and money the formula is not calibrated and validated.

Nevertheless, the information gathered within the project has been used to develop a quick-and-dirty formula which computes the breach growth as function of the soil characteristics of the dike and the actual hydraulic conditions over the breach (i.e. upstream and downstream water levels). This formula is presented hereafter. Only recently this physical-based formula was implemented into Delft1D2D. Presently computations are made for further validating this formula with field tests of a (cohesive) dike breach in Norway and laboratory experiments.
The formula assumes that an initial breach (Bo) is present, e.g. over a certain length the crest level has been lowered due to, for instance, a gully as a result of overflowing water or sliding of the inner slope. The process of sliding is well-known and occurs suddenly, but the development of gullies is less familiar. A limited review of available literature resulted on information on head cut migration, e.g. erosion against the flow direction. The development of a gully or head cut from the land-side of the dike crest towards the water-side of the dike crest goes with a speed of a few meters per hour depending on the cover layer of the crest. As the dike crest width varies between 2 and 5 m this means that in less than an hour an initial breach over the entire width of the dike crest might develop. Crucial is the development of an initial erosion spot at the land-side of the crest.

Hereunder the derived quick-and-dirty breach growth formula - i.e. Verheij-vdKnaap (2002) formula - is given, viz:

for $t_{start} < t \leq t_o$ yields:

$$B(t) = B_o \tag{B.1}$$

$$z(t) = z_{crest\text{-}level} - (z_{crest\text{-}level} - z_{min})*(t/t_o) \tag{B.2}$$

for $t > t_o$ {hence $B(t) \geq B_o$}:

$$B(t_i) = B(t_{i-1}) + \frac{\partial B}{\partial t}\Delta t \tag{B.3}$$

and $$\left(\frac{\partial B}{\partial t}\right)_{t_i} = \frac{f_1 f_2}{\ln 10}\frac{\{g(h_{up} - h_{down})\}^{1.5}}{u_c^2}\frac{1}{1 + \frac{f_2 g}{u_c}(t_i - t_0)} \tag{B.4}$$

conditions: $h_{down} \geq z_{min}$ else $h_{down} = z_{min}$
$B(t_i) \geq B(t_{i-1})$ else $B(t_i) = B(t_{i-1})$

in which:

f_1	: Constant factor [-]
f_2	: Constant factor [-]
B(t)	: Width of the dike-breach at point-in-time t [m]
B_o	: Initial width of the dike-breach [m]
g	: Acceleration due to gravity [m.s^{-2}]
h_{up}	: Upstream water level at point-in-time t [m]
h_{down}	: Downstream water level at point-in-time t [m]
t	: Actual computational point-in-time [hr]
t_o	: Computational point-in-time at which the width and the elevation of the dike-breach are respectively equal to B_o and z_{min} [hr]

t_{start} : Computational point-in-time at which the development of the breach starts [hr]

u_c : critical flow velocity [$m.s^{-1}$]

$z(t)$: Elevation of the dike-breach at point-in-time t [m]

$z_{crest\text{-}level}$: Elevation of the crest-level of the dike at $t=t_{start}$; initial crest level [m]

z_{min} : lowest level of the breach (level at $t=t_o$;input parameter) [m]

C Stochastic model parameters applied in the Case studies

C.1 Rules applied in determining possible failure due to overtopping or piping

The rules applied in SOBEK-Urban/Rural in determining if a failure due to overtopping or piping might occur are given in the algorithm below. The rules were applied for both Case study no 1 and Case study no 2.

```
Critical water level for piping  = Critical piping depth  + Local polder surface-level

IF ((Computed water level  > Critical water level for overtopping) “And” (Breach = 0))
       Breach = 1
ENDIF

IF ((Computed water level  > Critical water level for piping) “And” (Breach = 0))
       TimePipingCriteriaExceedance = TimePipingCriteriaExceedance + dt
ENDIF

IF (TimePipingCriteriaExceedance > Critical duration for exceedance of piping depth)
            Breach   = 1
ENDIF
```

Notes:

1. Breach is a Boolean value, Breach = 0 means no dike-breach; Breach = 1 means dike fails,
2. Computed water levels, Critical water level for piping, Local polder surface-level and Critical water level for overtopping are with respect to SOBEK-Rural/Urban datum,
3. Critical piping depth = Critical water level for piping with respect to a specific datum, that is equal to Local polder surface-level,
4. TimePinpingCriteraExceedance = Time period during which the critical piping water level was exceeded in the computation,
5. dt = actual computational time-step in SOBEK-Urban/Rural.

C.2 Probability density functions of stochastic model parameters

In Case study no 1 and Case study no 2, following six stochastic model parameters were considered, viz:

1. the critical water level for overtopping;
2. the critical water depth for piping;
3. the critical duration for exceedance of piping depth;
4. the duration of breach growth;
5. the final breach width; and
6. the discharge scaling factor.

The first five stochastic parameters yield for each and every dike-breach location, unless stated otherwise. The selected probability density functions (including mean, standard deviation or coefficient of variation) of these six stochastic parameters are given in Table C.1. Furthermore, hereunder additional information on these probability density functions is given.

Table C.1 Probability density functions for considered stochastic model parameters in Case study no 1 and Case study no 2.

Stochastic model parameter	**Probability density function**	**Mean**	**Coefficient of Variation (V) or Standard deviation (σ)**
Critical water level for overtopping (m)	Normal	12.12 [1)]	$\sigma = 0.3$
Critical water depth for piping (m)	Lognormal	4	V = 0.125
Critical duration for exceedence of piping depth (hrs)	Lognormal	6	V = 0.33
Duration of breach growth (hrs)	Lognormal	40	V = 0.5
Final breach width (m)	Lognormal	100	V = 0.5
Discharge scaling factor (-)	Exponential	0.3	$\sigma = 0.2$

Notes:
1. The critical water level for overtopping varies per dike-breach location and corresponds to a discharge with a return period of 90 years (see also Table 4.5)
2. Coefficient of variation is equal to mean divided by the standard deviation

(i) Critical water level for overtopping

The overtopping failure mechanism will occur as soon as computed water level exceed the critical water level for overtopping (i.e. the local dike height). The critical water level for overtopping is regarded as a normal random variable, its mean is equal to the local water level corresponding to a discharge of 9000 m^3/s, having a return period of 90 years. The coefficient of variation is equal to 0.3 m. This critical water level for overtopping is considered to include factors like wind speed, wave run up, resistance against erosion and so on. Of course, one could model these things explicitly, but that is not essential for the case study no 1 and case study no 2.

The distribution function for the dike height (or critical water level) is given by:

$$F(x) = \Phi \{ (x-\mu) / \sigma \} \qquad \text{(C.1)}$$

where Φ is the distribution function for the standard normal distribution (mean zero and unit standard deviation).

(ii) Critical water depth for piping and critical duration for exceedance of piping depth:

Piping occurs as a result of a difference in water pressure on both side of a dike. In Case study no 1 and Case study no 2, it is assumed that a breach will occur in case the critical water depth for piping h_p (i.e. measured with respect to the surface-level of the inner local polder area) is exceeded for a period longer than T_p (i.e. critical duration for exceedance of piping depth). The critical water depth for piping h_p is assumed to have a lognormal distribution with the mean value equal to 4 m and a coefficient of variation of 0.5 m. The critical duration for exceedance of piping depth (T_p) has a mean of 6 hours and a coefficient of variation of 2 hours.

(iii) Duration of breach growth and Final breach width

The growth-speed and final width of the dike-breach are random variables having a large uncertainty. As they are positive by nature, a normal distribution cannot be used. In those cases the lognormal distribution is quite appropriate. By definition a lognormal distribution for x means that ln x has a normal distribution, so

$$F(x) = \Phi \{ (\ln x - m) / s \} \qquad (C.2)$$

Now m and s are the mean and standard deviation of the natural logarithm of x. The means and standard deviation of x itself are given by:

$$\mu(x) = \exp (m + 0.5 s^2) \qquad (C.3)$$

$$V(x) = \sqrt{(\exp(s^2) - 1)} \approx s \qquad (C.4)$$

For the mean value of the final breach width a value of 100 m was assumed. For the mean value of the duration of breach growth a value 40 hours was assumed. For the coefficients of variation of both the final breach width and the duration of breach growth a value of 0.50 was taken.

(iv) Discharge scaling factor:

The discharge scaling factor applied to the peak discharge (i.e. 9000 m^3/s) of the upstream discharge hydrograph (see Fig 4.3) is assumed to be have an exponential distribution with a mean equal to 0.3 and a coefficient of variation equal to 0.2. The distribution function is given by:

$$F(x) = 1- \exp (-(x-a)/b) \qquad (C.5)$$

where F(x) is the distribution function for x and a en b are parameters

The exponential distribution may be considered as the most simplistic one of the extreme value distribution family, including distributions like Gumbel and Pareto that are being used in practice. The mean and coefficient of variation of the exponential distribution are respectively equal to (a+b) and b. As a result we have a = 0.1 and b = 0.2.

The dikes are designed for discharges having a scaling factor equal to 1.0. The probability that this design level x=1 is exceeded is equal to 0.011 or a return period of about 90 years.

$$1-F(x=1) = \exp (- (1.0 - 0.1)/0.2) = \exp (-4.5) = 0.011 \qquad (C.6)$$

The failure probability of dikes designed for the level x=1 will be slightly higher as we also have random properties on the resistance side.

D Detailed Results of Case study no 1

In this Appendix the flood-caused damage, the number of failed scenario's and resulting flood risks for Case study no 1 are given. In Table D.1 an overview is given of the flood damage information provided in Tables D.2 to D.4 is given. The flood risks are presented in section 5.1.

Table D.1 Overview of flood damage information given in Tables D.2 to D.4

Table	Failure mechanisms	Areas considered
D.2	Overtopping & Overflow	Northern polder only; Southern polder only; and entire catchment,
D.3	Piping only	Northern polder only; Southern polder only; and entire catchment,
D.4	Overtopping only	Northern polder only; Southern polder only; and entire catchment,

Table D.2. Case study no 1, Option 1, Flood damage per scenario in Euro, number of failed scenario's and Flood risk in 10^6 Euro on basis of 3000 scenario's, considering the overtopping and piping failure mechanism and probability density functions for model parameters as given in Table 4.4. Flood risks considering each polder individually and considering the entire catchment are given. Only scenario's in which damage occurred are given.

Flood Damage in Euro (Case study no 1; Option 1)						
		Entire catchment			Southern polder only	Northern polder only
No. of failed scenario's	SOBEK Case No.	Damage in Euro in Southern polder	Damage in Euro in Northern polder	Damage in Euro in Entire catchment	Damage in Euro in Southern polder	Damage in Euro in Northern polder
1	1	3.13E+09	1.20E+10	1.51E+10	3.13E+09	1.20E+10
2	2	7.12E+08	6.30E+09	7.01E+09	2.82E+09	6.48E+09
3	3	7.12E+08	5.92E+09	6.63E+09	2.82E+09	5.92E+09
4	4	2.75E+09	2.89E+09	5.64E+09	2.82E+09	2.89E+09
5	5	7.12E+08	2.89E+09	3.60E+09	2.75E+09	3.51E+09
6	6	0	5.42E+09	5.42E+09	2.82E+09	5.42E+09
7	7	7.12E+08	5.43E+09	6.14E+09	2.82E+09	5.43E+09
8	8	7.12E+08	5.43E+09	6.14E+09	2.82E+09	5.43E+09
9	9	7.12E+08	2.89E+09	3.60E+09	7.12E+08	2.89E+09
10	10	0	5.11E+09	5.11E+09	2.75E+09	5.11E+09
11	11	7.12E+08	2.89E+09	3.60E+09	7.12E+08	2.89E+09
12	12	7.12E+08	2.89E+09	3.60E+09	7.12E+08	2.89E+09
13	13	7.12E+08	2.67E+09	3.38E+09	7.12E+08	2.89E+09
14	14	0	2.89E+09	2.89E+09	7.04E+08	2.89E+09
15	15	0	2.89E+09	2.89E+09	7.09E+08	2.89E+09
16	16	0	2.57E+09	2.57E+09	0	2.57E+09
17	17	7.12E+08	1.70E+09	2.41E+09	7.12E+08	1.70E+09
18	18	0	2.81E+09	2.81E+09	7.12E+08	2.81E+09
19	22	0	9.78E+08	9.78E+08	0	9.78E+08
20	24	0	1.18E+09	1.18E+09	0	1.18E+09
21	27	0	0	0	3.13E+09	0
22	28	2.82E+09	4.81E+09	7.62E+09	2.82E+09	4.50E+09
23	29	2.20E+09	5.11E+09	7.31E+09	2.82E+09	5.11E+09
24	30	0	5.92E+09	5.92E+09	2.82E+09	5.92E+09
25	31	7.12E+08	2.79E+09	3.50E+09	7.12E+08	2.79E+09
26	32	7.12E+08	3.82E+09	4.53E+09	2.82E+09	4.50E+09
27	33	0	3.51E+09	3.51E+09	7.12E+08	3.51E+09

Flood Damage in Euro (Case study no 1; Option 1)						
		Entire catchment			**Southern polder only**	**Northern polder only**
No. of failed scenario's	SOBEK Case No.	Damage in Euro in Southern polder	Damage in Euro in Northern polder	Damage in Euro in Entire catchment	Damage in Euro in Southern polder	Damage in Euro in Northern polder
28	34	7.12E+08	2.89E+09	3.60E+09	7.12E+08	2.89E+09
29	35	7.12E+08	2.89E+09	3.60E+09	7.12E+08	2.89E+09
30	36	0	5.43E+09	5.43E+09	2.57E+09	5.43E+09
31	37	0	2.89E+09	2.89E+09	7.12E+08	2.89E+09
32	38	7.12E+08	2.88E+09	3.60E+09	7.10E+08	2.88E+09
33	39	7.12E+08	2.79E+09	3.50E+09	5.89E+08	2.79E+09
34	40	6.10E+08	2.87E+09	3.48E+09	7.12E+08	2.88E+09
35	41	1.59E+08	2.88E+09	3.04E+09	7.10E+08	2.88E+09
36	42	6.32E+08	2.70E+09	3.33E+09	7.10E+08	2.70E+09
37	43	6.09E+08	2.89E+09	3.50E+09	7.12E+08	2.89E+09
38	44	0	2.83E+09	2.83E+09	6.04E+08	2.83E+09
39	45	0	2.74E+09	2.74E+09	7.12E+08	2.74E+09
40	46	0	2.89E+09	2.89E+09	6.09E+08	2.89E+09
41	47	5.89E+08	2.70E+09	3.29E+09	1.37E+08	2.70E+09
42	48	0	2.82E+09	2.82E+09	7.08E+08	2.82E+09
43	49	0	2.67E+09	2.67E+09	0	2.67E+09
44	50	0	2.85E+09	2.85E+09	0	2.85E+09
45	51	0	2.74E+09	2.74E+09	6.32E+08	2.74E+09
46	52	6.32E+08	0	6.32E+08	6.08E+08	0
47	53	5.88E+08	2.28E+09	2.87E+09	0	2.31E+09
48	54	0	1.66E+09	1.66E+09	0	1.66E+09
49	55	0	2.17E+09	2.17E+09	5.62E+08	2.17E+09
50	56	4.71E+08	1.34E+09	1.82E+09	0	1.36E+09
51	57	0	2.36E+09	2.36E+09	0	2.36E+09
52	58	0	2.18E+09	2.18E+09	0	2.18E+09
53	59	0	2.60E+09	2.60E+09	0	2.60E+09
54	60	0	2.67E+09	2.67E+09	0	2.67E+09
55	61	0	8.83E+08	8.83E+08	0	8.83E+08
56	62	0	1.10E+09	1.10E+09	0	1.10E+09
57	63	0	1.92E+09	1.92E+09	0	1.92E+09
58	65	0	1.24E+09	1.24E+09	0	1.24E+09
59	72	0	5.66E+08	5.66E+08	0	5.66E+08
60	74	0	1.38E+09	1.38E+09	0	1.38E+09
61	76	0	1.37E+09	1.37E+09	0	1.37E+09
Total damage in €		2.66E+10	1.83E+11	2.09E+11	6.05E+10	1.84E+11
No. scenario's failed		28	59	60	42	59

Flood risk in 10^6 Euro (Case study no 1; Option 1)					
	Entire catchment			**Southern polder only**	**Northern polder only**
Number of scenario's	Southern polder	Northern polder	Entire catchment	Southern polder	Northern polder
3000	8.86	60.9	69.8	20.2	61.4

Table D.3. Case study no 1, Option 2, Flood damage per scenario in Euro, number of failed scenario's and Flood risk in 10^6 Euro on basis of 3000 scenario's, considering the piping failure mechanism only and probability density functions for model parameters as given in Table 4.4, except for model parameter "Critical water level for overtopping". Flood risks considering each polder individually and considering the entire catchment are given. Only scenario's in which damage occurred are given.

Flood Damage in Euro (Case study no 1; Option 2)						
		Entire Catchment			**Southern polder only**	**Northern polder only**
No. of failed scenario's	SOBEK Case No.	Damage in Euro in Southern polder	Damage in Euro in Northern polder	Damage in Euro in Entire Catchment	Damage in Euro in Southern polder	Damage in Euro in Northern polder
1	1	3.13E+09	1.19E+10	1.50E+10	3.13E+09	1.19E+10
2	2	0	6.86E+09	6.86E+09	2.82E+09	6.86E+09
3	3	7.12E+08	5.43E+09	6.14E+09	7.12E+08	5.43E+09
4	4	0	2.88E+09	2.88E+09	0	2.88E+09
5	5	0	5.43E+09	5.43E+09	7.12E+08	5.43E+09
6	6	0	5.43E+09	5.43E+09	7.12E+08	5.43E+09
7	7	0	6.55E+09	6.55E+09	7.12E+08	6.55E+09
8	8	7.12E+08	5.55E+09	6.26E+09	7.12E+08	5.55E+09
9	9	7.09E+08	2.89E+09	3.60E+09	7.12E+08	2.89E+09
10	10	0	5.30E+09	5.30E+09	7.12E+08	5.30E+09
11	11	0	2.89E+09	2.89E+09	0	2.89E+09
12	12	0	2.86E+09	2.86E+09	0	2.86E+09
13	13	7.12E+08	0	7.12E+08	7.12E+08	0
14	14	0	2.89E+09	2.89E+09	6.01E+08	2.89E+09
15	15	0	2.87E+09	2.87E+09	0	2.87E+09
16	16	0	2.56E+09	2.56E+09	0	2.56E+09
17	17	6.77E+08	1.37E+09	2.04E+09	7.04E+08	1.46E+09
18	22	0	3.70E+08	3.70E+08	0	3.70E+08
19	24	0	1.14E+09	1.14E+09	0	1.14E+09
20	28	2.82E+09	2.89E+09	5.71E+09	2.82E+09	2.89E+09
21	29	0	3.20E+09	3.20E+09	1.21E+09	3.20E+09
22	30	7.12E+08	3.82E+09	4.53E+09	2.75E+09	4.50E+09
23	31	7.12E+08	2.65E+09	3.36E+09	7.12E+08	2.65E+09
24	32	7.12E+08	4.81E+09	5.52E+09	7.12E+08	4.81E+09
25	33	0	5.43E+09	5.43E+09	7.12E+08	5.43E+09
26	34	0	3.51E+09	3.51E+09	0	3.51E+09
27	35	7.08E+08	2.45E+09	3.15E+09	7.12E+08	2.46E+09
28	36	0	2.89E+09	2.89E+09	0	2.89E+09
29	37	0	2.89E+09	2.89E+09	0	2.89E+09
30	38	0	2.69E+09	2.69E+09	0	2.69E+09
31	39	0	2.73E+09	2.73E+09	0	2.73E+09
32	40	0	2.61E+09	2.61E+09	0	2.61E+09
33	41	0	2.86E+09	2.86E+09	0	2.86E+09
34	42	0	2.75E+09	2.75E+09	0	2.75E+09
35	43	0	2.89E+09	2.89E+09	0	2.89E+09
36	44	0	2.75E+09	2.75E+09	0	2.75E+09
37	45	0	2.68E+09	2.68E+09	0	2.68E+09
38	46	0	2.89E+09	2.89E+09	0	2.89E+09

Flood Damage in Euro (Case study no 1; Option 2)						
		Entire Catchment			**Southern polder only**	**Northern polder only**
No. of failed scenario's	SOBEK Case No.	Damage in Euro in Southern polder	Damage in Euro in Northern polder	Damage in Euro in Entire Catchment	Damage in Euro in Southern polder	Damage in Euro in Northern polder
39	47	0	2.73E+09	2.73E+09	0	2.73E+09
40	48	0	2.87E+09	2.87E+09	9.68E+07	2.87E+09
41	50	0	2.84E+09	2.84E+09	0	2.84E+09
42	51	0	2.73E+09	2.73E+09	0	2.73E+09
43	53	0	2.36E+09	2.36E+09	0	2.36E+09
44	54	0	1.58E+09	1.58E+09	0	1.58E+09
45	55	0	1.88E+09	1.88E+09	0	1.88E+09
46	56	0	1.25E+09	1.25E+09	0	1.25E+09
47	57	0	2.33E+09	2.33E+09	0	2.33E+09
48	58	0	2.18E+09	2.18E+09	0	2.18E+09
49	59	0	2.61E+09	2.61E+09	0	2.61E+09
50	61	0	7.05E+08	7.05E+08	0	7.05E+08
51	62	0	1.07E+09	1.07E+09	0	1.07E+09
52	65	0	1.23E+09	1.23E+09	0	1.23E+09
53	72	0	4.96E+08	4.96E+08	0	4.96E+08
54	74	0	1.29E+09	1.29E+09	0	1.29E+09
55	76	0	1.36E+09	1.36E+09	0	1.36E+09
Total Damage in €		1.23E+10	1.66E+11	1.78E+11	2.27E+10	1.67E+11
No. Scenario's failed		11	54	55	20	54
Flood risk in 10^6 Euro (Case study no 1; Option 2)						
		Entire catchment			**Southern polder only**	**Northern polder only**
Number of scenario's		Southern polder	Northern polder	Entire Catchment	Southern polder	Northern polder
3000		4.10	55.34	59.44	7.56	55.61

Table D.4. Case study no 1, Option 3, Flood damage per scenario in Euro, Number of failed scenario's and Flood risk in 10^6 Euro on basis of 1000 scenario's, considering the overtopping failure mechanism only and probability density functions for model parameters as given in Table 4.4, except for model parameter "Critical water depth for piping" and "Critical duration for exceedance of piping depth". Flood risks considering each polder individually and considering the entire catchment are given. Only scenario's in which damage occurred are given.

Flood Damage in Euro (Case study no 1; Option 3)						
		Entire Catchment			**Southern polder only**	**Northern polder only**
No. of failed scenario's	SOBEK Case No.	Damage in Euro in Southern polder	Damage in Euro in Northern polder	Damage in Euro in Entire catchment	Damage in Euro in Southern polder	Damage in Euro in Northern polder
1	1	3.13E+09	1.20E+10	1.51E+10	3.13E+09	1.20E+10
2	2	7.12E+08	6.42E+09	7.13E+09	2.82E+09	6.55E+09
3	3	7.12E+08	6.05E+09	6.76E+09	2.82E+09	6.17E+09
4	4	2.82E+09	2.89E+09	5.71E+09	2.82E+09	2.89E+09
5	5	7.12E+08	2.89E+09	3.60E+09	2.82E+09	3.51E+09
6	6	0	5.42E+09	5.42E+09	2.82E+09	5.42E+09
7	7	7.12E+08	5.43E+09	6.14E+09	2.82E+09	5.43E+09
8	8	7.12E+08	5.42E+09	6.14E+09	2.82E+09	5.43E+09
9	9	7.12E+08	2.89E+09	3.60E+09	7.12E+08	2.89E+09
10	10	0	5.30E+09	5.30E+09	2.82E+09	5.30E+09
11	11	7.12E+08	2.89E+09	3.60E+09	7.12E+08	2.89E+09
12	12	7.12E+08	2.88E+09	3.60E+09	7.12E+08	2.89E+09
13	13	7.12E+08	2.89E+09	3.60E+09	7.12E+08	2.89E+09
14	14	6.09E+08	2.87E+09	3.48E+09	7.10E+08	2.87E+09
15	15	0	2.89E+09	2.89E+09	7.12E+08	2.89E+09
16	16	0	1.50E+09	1.50E+09	0	1.50E+09
17	17	7.12E+08	1.83E+09	2.54E+09	7.12E+08	1.86E+09
18	18	0	2.87E+09	2.87E+09	7.12E+08	2.87E+09
19	22	0	7.25E+08	7.25E+08	0	7.25E+08
Total Damage in €		1.37E+10	7.61E+10	8.97E+10	3.14E+10	7.70E+10
No. Scenario's failed		13	19	19	17	19
Flood risk in 10^6 Euro (Case study no 1; Option 3)						
		Entire catchment			**Southern polder only**	**Northern polder only**
Number of scenario's		Southern polder	Northern polder	Entire catchment	Southern polder	Northern polder
1000		13.7	76.0	89.7	31.4	77.0

E Detailed Results of Case study no 2

In this Appendix the flood-caused damage, the number of failed scenario's and resulting flood risks for Case study no 2 are given. In Table E.1 an overview is given of the flood damage information provided in Tables E.3 to E.8. The flood risks are presented in section 5.2.3.

In the Monte Carlo analysis a number of different flood scenario's are determined by conducting jointly several random samplings on the probability density functions of considered stochastic model parameters (see Table 4.4). As an example the result of 84 of such samplings for the reference situation no a (i.e. Ref a, see section 5.2.1) is given in Table E.2

Table E.1 Overview of flood damage information given in Tables E.3 to E.8

Table	Failure mechanisms	Simulation	Areas considered
E.3	Overtopping & piping	Reference a	Entire catchment
E.4	Overtopping only	Reference b	Entire catchment
E.5	Overtopping & piping	1a (fixed weir)	Entire catchment
E.6	Overtopping only	1b (fixed weir)	Entire catchment
E.7	Overtopping & piping	2a (adjustable weir)	Entire catchment
E.8	Overtopping only	2b (adjustable weir)	Entire catchment

Table E.2 Results of 84 samplings on probability density functions given in Table 4.4 for reference situation no a (i.e. Ref a, see section 5.2.1)

Nr	Samp-ling No.	Alfa	Q_{peak}	Dike1	Dike2	Dike3	Dike4	Dike5	Dike6	P1	P2	P3	P4	P5	P6
Mean		1	9000	12.53	11.87	10.30	11.47	13.19	11.77	4	4	4	4	4	4
1	699	1.85	16606	12.15	11.71	10.38	11.54	13.32	11.94	4.13	4.57	4.34	4.77	4.54	4.16
2	1130	1.38	12464	12.86	12.32	10.70	11.45	12.84	11.95	4.73	4.54	4.09	3.96	4.11	4.36
3	1864	1.36	12274	12.58	11.31	10.52	11.73	13.39	11.79	3.16	4.49	3.73	4.18	3.31	4.06
4	630	1.34	12051	12.45	11.62	10.01	11.42	13.86	11.40	3.41	4.52	4.02	3.66	3.63	4.06
5	1552	1.28	11535	11.90	11.53	10.09	11.40	12.70	11.46	3.70	3.39	3.99	3.84	3.75	4.43
6	1157	1.27	11452	12.80	11.62	10.09	11.61	13.27	12.03	3.85	4.01	4.39	4.73	3.80	3.63
7	369	1.22	11008	12.19	11.71	10.37	11.96	12.98	11.99	3.68	4.38	4.63	3.70	5.16	4.11
8	1268	1.22	11006	12.53	11.85	10.63	11.69	13.37	11.80	3.83	3.80	3.76	4.25	3.67	3.45
9	1708	1.17	10490	12.72	11.69	9.91	11.82	13.16	11.57	4.14	4.31	3.82	3.81	4.07	4.31
10	1920	1.16	10478	12.42	12.08	10.31	11.78	13.30	11.82	3.87	3.51	4.24	4.00	3.71	3.98
11	299	1.15	10314	12.29	12.06	10.81	11.68	13.41	11.61	4.10	3.94	4.44	4.37	3.84	4.06
12	1363	1.14	10254	12.32	12.10	10.11	11.33	13.01	11.74	3.59	4.00	3.50	4.56	4.06	3.95
13	1635	1.12	10062	12.73	11.61	9.71	11.46	13.21	12.09	3.91	3.54	4.12	4.38	4.64	4.38
14	2803	1.11	9947	12.90	12.01	9.95	11.64	13.80	11.86	4.81	4.80	3.98	4.14	4.03	4.18
15	147	1.10	9938	11.89	11.70	10.39	11.44	13.05	11.55	4.79	3.93	5.42	4.42	3.49	3.87
16	2163	1.09	9825	12.55	11.71	10.13	10.91	13.20	11.22	6.17	3.36	4.46	4.00	4.59	2.78
17	868	1.08	9708	11.80	12.19	10.44	10.71	13.01	11.80	4.24	3.69	4.76	4.21	3.92	4.01
18	11	1.08	9682	11.97	11.66	10.12	11.83	12.94	11.54	4.60	4.76	4.40	4.10	4.09	3.97
19	2680	1.07	9665	12.07	11.69	10.56	11.24	13.01	11.58	3.81	4.39	3.91	4.55	3.41	4.14
20	704	1.07	9602	11.84	12.16	10.22	11.62	13.41	11.46	5.10	4.16	4.25	4.37	4.00	3.79
21	878	1.05	9449	12.46	11.77	10.18	11.10	12.99	11.57	3.97	4.36	3.69	3.93	3.59	5.08
22	67	1.05	9427	12.56	11.59	9.91	11.72	12.90	11.69	3.91	4.16	4.35	3.32	3.54	3.95
23	1101	1.04	9332	11.96	11.71	10.22	11.82	12.81	11.55	3.61	3.24	4.06	3.77	3.82	3.87

Nr	Samp-ling No.	Alfa	Q_{peak}	Dike1	Dike2	Dike3	Dike4	Dike5	Dike6	P1	P2	P3	P4	P5	P6
24	1754	1.02	9219	12.96	11.53	10.24	11.82	13.31	11.87	3.19	4.14	3.41	3.60	3.99	3.98
25	1739	1.02	9181	12.45	12.01	10.32	11.66	13.25	12.27	3.83	4.35	3.35	3.96	3.35	4.07
26	461	1.02	9137	12.03	11.75	10.36	11.84	12.63	11.84	3.25	3.81	3.45	3.92	4.18	3.54
27	2724	1.00	9013	12.00	11.70	10.13	11.65	12.91	11.51	5.35	3.14	4.02	3.42	3.83	4.31
28	1457	0.99	8881	12.75	11.62	10.43	11.28	12.78	11.93	4.14	3.66	3.74	3.50	3.86	3.95
29	1233	0.98	8778	12.31	11.52	9.94	11.95	13.60	12.14	4.39	3.37	4.03	4.36	3.35	3.54
30	1096	0.97	8715	12.92	11.73	11.05	11.33	12.88	11.59	4.55	4.61	4.36	3.22	4.09	3.91
31	1802	0.97	8685	12.74	12.00	10.52	11.47	12.90	12.05	4.54	4.86	3.33	4.09	3.68	3.29
32	2613	0.95	8565	12.71	11.70	10.60	12.08	13.34	11.40	3.58	4.08	4.08	3.27	3.71	4.81
33	682	0.93	8400	11.85	11.52	10.33	11.39	13.17	12.13	3.79	3.78	3.62	5.15	3.47	3.50
34	356	0.91	8222	12.20	12.20	10.22	11.27	12.05	11.72	4.06	3.86	4.46	3.81	4.48	3.77
35	1210	0.90	8105	12.77	12.10	10.29	11.31	13.02	11.14	3.91	3.91	3.14	3.27	4.22	3.43
36	2754	0.86	7722	12.92	12.21	9.12	11.55	12.56	11.84	4.06	4.54	4.62	4.77	3.31	4.27
37	808	0.96	8664	12.53	12.11	10.16	11.66	13.03	11.80	4.26	3.17	4.59	4.62	3.95	3.98
38	2028	0.94	8504	12.79	11.66	10.02	11.71	13.53	11.95	3.32	5.24	4.04	3.69	4.27	3.75
39	1214	0.94	8501	12.46	12.15	10.66	11.66	13.13	12.28	3.11	4.13	3.81	3.95	3.78	4.13
40	254	0.93	8384	12.76	11.64	9.95	11.76	12.98	11.48	3.99	4.24	4.60	3.37	4.26	3.51
41	1565	0.92	8280	12.41	11.83	10.53	11.24	13.11	11.52	4.03	3.86	4.39	4.64	3.99	5.15
42	2242	0.91	8224	12.47	11.66	10.26	11.73	13.38	11.92	3.59	3.41	4.76	3.55	3.62	4.80
43	2032	0.91	8206	12.53	11.93	10.58	11.29	13.23	11.82	3.78	4.17	3.95	4.62	4.08	4.27
44	1150	0.91	8146	12.58	11.48	10.22	12.01	12.96	11.54	3.63	4.83	3.29	3.36	4.09	3.60
45	2651	0.90	8138	12.48	12.03	10.96	11.42	12.83	11.88	3.34	3.66	4.89	4.47	3.98	3.97
46	2076	0.90	8072	12.71	11.89	10.11	11.50	13.41	11.65	3.52	3.49	4.54	3.55	4.28	4.89
47	2787	0.88	7948	12.08	12.12	10.59	11.47	12.95	11.77	4.10	4.96	4.03	4.11	3.51	4.20
48	225	0.88	7930	12.93	11.70	10.49	11.59	13.17	11.40	3.39	3.44	4.25	4.11	3.96	5.04
49	2983	0.87	7875	12.40	12.05	10.77	11.64	12.75	11.64	2.87	3.20	4.21	4.08	3.55	3.68
50	1582	0.87	7844	12.25	11.37	9.79	11.50	13.13	11.45	4.65	4.07	3.50	4.31	3.91	3.62
51	1983	0.87	7837	12.42	11.22	10.60	11.28	13.63	11.64	4.87	4.35	4.38	3.61	3.99	4.30
52	2820	0.87	7792	12.59	12.05	10.32	11.40	12.85	11.68	4.31	3.47	3.27	3.64	3.65	4.64
53	2060	0.86	7780	12.56	11.85	10.47	11.53	13.30	11.58	2.71	3.77	3.69	3.60	3.37	4.24
54	1891	0.86	7772	13.05	11.47	10.60	11.31	13.06	11.80	3.94	3.20	3.28	3.16	4.24	4.48
55	2338	0.86	7752	11.94	12.24	10.56	10.91	13.34	11.48	3.56	3.31	3.89	3.28	3.81	3.50
56	988	0.85	7661	12.30	11.98	10.96	11.47	13.77	11.65	4.23	4.60	4.25	3.34	4.22	3.67
57	233	0.85	7651	12.26	11.71	10.25	11.74	13.12	11.58	3.92	4.13	6.02	4.82	3.29	4.17
58	2594	0.85	7645	12.44	11.54	10.44	11.30	13.48	11.82	4.06	3.82	3.87	4.23	3.63	4.66
59	1145	0.84	7604	12.28	12.50	10.06	11.60	13.43	12.00	3.83	3.98	3.99	3.72	4.00	4.15
60	1479	0.84	7600	12.35	11.97	10.05	11.29	13.35	11.71	4.02	4.88	3.57	3.99	4.01	3.94
61	2262	0.84	7593	12.86	11.40	10.22	11.38	12.65	12.46	5.04	3.71	3.78	4.05	4.80	3.37
62	858	0.84	7551	12.12	11.93	10.41	11.82	13.30	11.77	3.43	5.13	5.14	4.11	4.43	4.42
63	2511	0.84	7528	12.91	11.80	10.51	11.05	13.55	11.48	3.75	3.35	4.68	3.84	4.34	4.07
64	2247	0.83	7514	12.36	11.65	10.16	11.41	13.10	11.72	4.40	3.72	3.61	3.70	3.42	3.53
65	1345	0.83	7502	12.64	11.66	10.26	11.84	12.71	12.06	3.81	3.88	4.33	3.18	3.30	4.49
66	1428	0.83	7451	12.68	11.09	10.33	11.67	12.93	11.61	3.91	3.72	4.30	3.27	3.56	4.14
67	1531	0.83	7447	12.80	11.54	9.89	11.48	13.19	12.02	4.85	4.51	4.31	4.73	4.09	3.46
68	1510	0.82	7375	12.48	11.98	10.45	11.66	13.37	11.97	3.26	4.43	3.55	3.78	4.05	3.39
69	2174	0.81	7313	12.17	11.41	10.16	11.91	12.91	11.56	4.26	4.27	3.62	3.37	4.23	4.07
70	556	0.81	7283	12.56	11.57	10.03	11.86	13.03	11.52	4.79	3.76	4.24	2.80	3.41	4.73
71	672	0.80	7187	12.25	11.86	10.70	11.46	13.29	11.09	3.52	3.23	4.23	3.83	4.93	3.58
72	238	0.79	7151	12.07	12.05	10.34	11.45	13.02	11.73	3.26	4.85	4.29	3.89	4.16	3.28
73	1957	0.79	7133	12.34	12.09	10.81	11.34	13.38	11.99	4.35	3.62	4.94	3.61	3.81	3.30

Nr	Samp-ling No.	Alfa	Q_{peak}	Dike1	Dike2	Dike3	Dike4	Dike5	Dike6	P1	P2	P3	P4	P5	P6
74	305	0.79	7098	12.41	11.59	10.30	11.20	13.16	11.31	4.40	4.59	4.33	3.65	4.23	3.56
75	978	0.79	7090	12.14	12.17	9.82	11.15	13.34	12.12	4.16	4.73	4.06	4.08	4.62	3.20
76	1045	0.78	7023	12.64	11.69	10.00	12.05	13.38	11.80	4.51	4.53	4.20	3.73	3.32	3.70
77	2258	0.77	6920	12.72	12.03	10.38	11.55	12.84	12.42	3.56	5.45	4.55	4.74	4.88	3.08
78	2097	0.77	6889	12.53	12.05	9.93	11.34	13.01	11.72	3.85	3.61	3.49	3.53	3.51	3.19
79	1973	0.77	6885	12.36	11.32	10.33	11.57	13.45	11.80	3.46	3.30	3.06	3.57	4.17	2.99
80	269	0.76	6872	12.76	12.01	10.62	11.17	13.03	11.44	3.81	4.82	3.79	3.62	3.12	3.83
81	1267	0.75	6789	12.33	11.95	10.55	11.55	13.45	11.10	3.58	3.17	3.29	3.79	3.85	3.31
82	587	0.75	6757	12.93	11.84	10.12	11.78	13.31	11.82	4.49	3.41	3.83	4.14	3.86	3.12
83	2409	0.73	6570	12.96	11.67	10.37	11.54	13.63	11.85	3.84	3.69	3.37	2.78	4.04	4.40
84	961	0.69	6178	12.06	11.89	10.40	11.63	13.59	11.62	4.30	3.85	4.00	4.07	2.70	3.19

Notes:
1. Alfa = discharge scale factor,
2. DikeX = the critical level for overtopping in metres above National datum (i.e. NAP) at breach location no. X,
3. PX = Critical level for Piping measured in m above the surrounding polder area at breach location X.

Table E.3 Case study no 2, Ref a, Flood damage per scenario in Euro and Flood risk in 10^6 Euro on basis of 3000 scenario's, considering the overtopping and piping failure mechanism and probability density functions for model parameters as given in Table 4.4. Flood risks considering the entire catchment are given. Only scenario's in which damage occurred are given.

Flood Damage in Euro *(*Case study no 2; Ref a)					
		Entire catchment			
No. of failed scenario's	SOBEK Case No.	Damage in Euro in Southern polder	Damage in Euro in Northern polder	Damage in Euro in Emergency Retention polder	Damage in Euro in Entire catchment
1	1	3.05E+09	1.18E+10	7.47E+07	1.49E+10
2	2	2.99E+09	5.43E+09	7.13E+07	8.49E+09
3	3	7.59E+08	5.55E+09	6.64E+07	6.38E+09
4	4	7.59E+08	5.43E+09	7.33E+07	6.26E+09
5	5	2.99E+09	5.43E+09	7.17E+07	8.49E+09
6	6	2.99E+09	5.30E+09	6.39E+07	8.35E+09
7	7	2.99E+09	5.43E+09	6.55E+07	8.48E+09
8	8	0.00E+00	5.43E+09	6.76E+07	5.49E+09
9	9	7.59E+08	2.79E+09	6.25E+07	3.61E+09
10	10	7.59E+08	2.71E+09	6.23E+07	3.54E+09
11	11	7.59E+08	2.84E+09	6.19E+07	3.66E+09
12	12	2.49E+09	2.46E+09	6.30E+07	5.02E+09
13	13	7.59E+08	2.22E+09	6.48E+07	3.04E+09
14	14	1.19E+09	1.16E+09	6.23E+07	2.41E+09
15	15	0.00E+00	5.30E+09	6.23E+07	5.36E+09
16	16	7.59E+08	1.60E+09	6.24E+07	2.42E+09
17	17	7.59E+08	2.68E+09	6.02E+07	3.50E+09
18	18	7.59E+08	2.85E+09	6.32E+07	3.67E+09
19	19	0.00E+00	2.30E+09	6.46E+07	2.36E+09
20	20	0.00E+00	2.58E+09	6.46E+07	2.64E+09
21	21	7.59E+08	2.43E+09	6.83E+07	3.26E+09
22	22	1.19E+09	2.33E+09	6.23E+07	3.58E+09
23	23	7.59E+08	2.67E+09	6.08E+07	3.49E+09
24	24	7.59E+08	2.65E+09	6.35E+07	3.47E+09

Flood Damage in Euro (Case study no 2; Ref a)					
		Entire catchment			
No. of failed scenario's	SOBEK Case No.	Damage in Euro in Southern polder	Damage in Euro in Northern polder	Damage in Euro in Emergency Retention polder	Damage in Euro in Entire catchment
25	25	0.00E+00	1.43E+09	7.08E+07	1.50E+09
26	26	0.00E+00	2.58E+09	6.23E+07	2.65E+09
27	27	7.59E+08	2.43E+09	6.23E+07	3.25E+09
28	28	7.59E+08	2.11E+09	5.68E+07	2.93E+09
29	29	0.00E+00	1.03E+09	6.23E+07	1.09E+09
30	30	7.56E+08	0.00E+00	6.23E+07	8.18E+08
31	31	0.00E+00	9.16E+08	6.53E+07	9.81E+08
32	32	7.59E+08	2.30E+09	5.20E+07	3.11E+09
33	33	0.00E+00	2.07E+09	6.23E+07	2.13E+09
34	34	0.00E+00	1.78E+09	5.66E+07	1.83E+09
35	35	7.59E+08	7.94E+08	3.38E+07	1.59E+09
36	36	0.00E+00	7.94E+08	6.08E+07	8.55E+08
37	37	0.00E+00	1.95E+09	3.15E+07	1.98E+09
38	38	7.59E+08	1.96E+09	3.76E+07	2.75E+09
39	39	0.00E+00	2.62E+09	6.12E+07	2.68E+09
40	40	7.59E+08	0.00E+00	3.47E+07	7.94E+08
41	41	0.00E+00	2.32E+09	5.40E+07	2.37E+09
42	42	7.59E+08	2.02E+09	5.35E+07	2.83E+09
43	43	0.00E+00	1.87E+09	0.00E+00	1.87E+09
44	44	7.52E+08	1.43E+09	3.99E+07	2.22E+09
45	45	0.00E+00	2.24E+09	0.00E+00	2.24E+09
46	46	7.52E+08	2.14E+09	0.00E+00	2.89E+09
47	47	0.00E+00	0.00E+00	6.23E+07	6.23E+07
48	48	0.00E+00	2.54E+09	0.00E+00	2.54E+09
49	49	0.00E+00	2.71E+09	0.00E+00	2.71E+09
50	50	0.00E+00	9.35E+08	3.08E+07	9.65E+08
51	51	7.52E+08	0.00E+00	0.00E+00	7.52E+08
52	52	7.52E+08	1.12E+09	6.23E+07	1.94E+09
53	53	0.00E+00	2.30E+09	3.75E+07	2.34E+09
54	54	7.56E+08	1.04E+09	0.00E+00	1.79E+09
55	55	7.57E+08	1.23E+09	3.34E+07	2.02E+09
56	56	7.55E+08	0.00E+00	0.00E+00	7.55E+08
57	57	0.00E+00	0.00E+00	6.23E+07	6.23E+07
58	58	0.00E+00	0.00E+00	2.13E+07	2.13E+07
59	60	0.00E+00	7.88E+08	0.00E+00	7.88E+08
60	61	0.00E+00	0.00E+00	2.47E+07	2.47E+07
61	62	0.00E+00	6.10E+08	0.00E+00	6.10E+08
62	63	0.00E+00	9.23E+08	0.00E+00	9.23E+08
63	64	0.00E+00	0.00E+00	6.00E+07	6.00E+07
64	65	7.59E+08	0.00E+00	2.38E+07	7.83E+08
65	66	7.47E+08	0.00E+00	0.00E+00	7.47E+08
66	68	0.00E+00	8.55E+08	2.56E+07	8.81E+08
67	69	7.59E+08	0.00E+00	0.00E+00	7.59E+08
68	70	7.59E+08	0.00E+00	0.00E+00	7.59E+08
69	71	0.00E+00	9.70E+08	0.00E+00	9.70E+08
70	75	0.00E+00	0.00E+00	2.56E+07	2.56E+07
71	77	0.00E+00	0.00E+00	9.37E+06	9.37E+06

Flood Damage in Euro (Case study no 2; Ref a)					
		Entire catchment			
No. of failed scenario's	SOBEK Case No.	Damage in Euro in Southern polder	Damage in Euro in Northern polder	Damage in Euro in Emergency Retention polder	Damage in Euro in Entire catchment
72	79	0.00E+00	3.83E+08	2.17E+07	4.04E+08
73	83	2.83E+08	0.00E+00	0.00E+00	2.83E+08
Total Damage in €		4.36E+10	1.45E+11	3.08E+09	1.91E+11
No. Scenario's failed		40	57	57	73
Flood risks in 10^6 Euro (Case study no 2; Ref a)					
		Entire catchment			
Number of scenario's		Southern polder	Northern polder	Emergency retention polder	Entire catchment
3000		14.6	48.2	1.0	63.7

Table E.4 Case study no 2, Ref b, Flood damage per scenario in Euro and Flood risk in 10^6 Euro on basis of 3000 scenario's, considering the overtopping failure mechanism only and probability density functions for model parameters as given in Table 4.4, except for model parameters "Critical water depth for piping" and "Critical duration for exceedance of piping depth". Flood risks considering the entire catchment are given. Only scenario's in which damage occurred are given.

Flood Damage in Euro (Case study no 2; Ref b)					
		Entire catchment			
No. of failed scenario's	SOBEK Case No	Damage in Euro in Southern polder	Damage in Euro in Northern polder	Damage in Euro in Emergency Retention polder	Damage in Euro in Entire catchment
1	1	3.05E+09	1.18E+10	7.47E+07	1.49E+10
2	2	3.32E+09	2.65E+09	7.36E+07	6.04E+09
3	3	0	6.30E+09	6.80E+07	6.37E+09
4	4	7.59E+08	5.43E+09	6.51E+07	6.25E+09
5	5	0	5.43E+09	7.26E+07	5.50E+09
6	6	2.99E+09	4.80E+09	6.39E+07	7.85E+09
7	7	0	5.43E+09	6.85E+07	5.49E+09
8	8	0	4.44E+09	6.78E+07	4.50E+09
9	9	0	2.67E+09	6.46E+07	2.74E+09
10	10	0	2.70E+09	6.23E+07	2.76E+09
11	11	0	2.84E+09	6.15E+07	2.90E+09
12	12	2.99E+09	2.31E+09	6.44E+07	5.37E+09
13	13	7.59E+08	1.51E+09	6.76E+07	2.34E+09
14	14	7.59E+08	1.44E+09	5.06E+07	2.25E+09
15	15	0	5.42E+09	0	5.42E+09
16	16	7.59E+08	1.46E+09	6.27E+07	2.28E+09
17	17	1.63E+09	2.67E+09	5.07E+07	4.35E+09
18	18	0	4.11E+09	0	4.11E+09
19	19	7.59E+08	2.36E+09	6.23E+07	3.18E+09
20	20	0	2.63E+09	5.10E+07	2.68E+09
21	21	7.59E+08	0	6.53E+07	8.24E+08
22	22	0	1.40E+09	6.23E+07	1.47E+09
23	23	0	2.57E+09	6.23E+07	2.63E+09
24	24	0	1.47E+09	0	1.47E+09

Flood Damage in Euro (Case study no 2; Ref b)					
		Entire catchment			
No. of failed scenario's	SOBEK Case No	Damage in Euro in Southern polder	Damage in Euro in Northern polder	Damage in Euro in Emergency Retention polder	Damage in Euro in Entire catchment
25	25	0	2.59E+09	0	2.59E+09
26	26	0	2.21E+09	6.39E+07	2.27E+09
27	27	0	2.33E+09	6.23E+07	2.39E+09
28	28	7.53E+08	1.10E+09	3.65E+07	1.89E+09
29	29	0	2.45E+09	0	2.45E+09
30	30	0	0	6.23E+07	6.23E+07
31	31	0	0	6.32E+07	6.32E+07
32	32	0	0	3.21E+07	3.21E+07
33	33	0	1.83E+09	0	1.83E+09
34	34	0	0	6.08E+07	6.08E+07
35	36	0	8.63E+08	0	8.63E+08
Total Damage in €		1.93E+10	9.72E+10	1.72E+09	1.18E+11
No. Scenario's failed		12	30	28	35
Flood risks in 10^6 Euro (Case study no 2; Ref b)					
		Entire catchment			
Number of scenario's		Southern polder	Northern polder	Emergency retention polder	Entire catchment
3000		6.43	32.38	0.57	39.39

Table E.5 Case study no 2, Option 1a (fixed weir), Flood damage per scenario in Euro and Flood risk in 10^6 Euro on basis of 3000 scenario's, considering the overtopping and piping failure mechanism and probability density functions for model parameters as given in Table 4.4. Flood risks considering the entire catchment are given. Only scenario's in which damage occurred are given.

Flood Damage in Euro (Case study no 2; Option 1a)					
		Entire catchment			
No. of failed scenario's	SOBEK Case No	Damage in Euro in Southern polder	Damage in Euro in Northern polder	Damage in Euro in Emergency Retention polder	Damage in Euro in Entire catchment
1	1	3.32E+09	1.18E+10	7.72E+07	1.52E+10
2	2	3.12E+09	3.41E+09	7.34E+07	6.60E+09
3	3	7.59E+08	5.49E+09	7.22E+07	6.32E+09
4	4	7.59E+08	5.43E+09	6.99E+07	6.26E+09
5	5	2.99E+09	5.42E+09	6.37E+07	8.48E+09
6	6	0	4.78E+09	6.80E+07	4.85E+09
7	7	2.99E+09	2.63E+09	6.13E+07	5.68E+09
8	8	0	5.42E+09	6.23E+07	5.49E+09
9	9	7.59E+08	2.79E+09	5.39E+07	3.61E+09
10	10	7.59E+08	2.76E+09	5.85E+07	3.57E+09
11	11	7.59E+08	2.84E+09	5.16E+07	3.65E+09
12	12	0	2.50E+09	5.97E+07	2.56E+09
13	13	0	2.24E+09	5.65E+07	2.30E+09
14	14	2.80E+09	1.19E+09	5.24E+07	4.04E+09
15	15	0	5.42E+09	4.68E+06	5.43E+09
16	16	7.59E+08	1.59E+09	4.97E+07	2.40E+09
17	17	7.59E+08	2.72E+09	1.44E+07	3.49E+09
18	18	7.59E+08	3.48E+09	1.64E+06	4.24E+09

Flood Damage in Euro (Case study no 2; Option 1a)					
		Entire catchment			
No. of failed scenario's	SOBEK Case No	Damage in Euro in Southern polder	Damage in Euro in Northern polder	Damage in Euro in Emergency Retention polder	Damage in Euro in Entire catchment
19	19	0	2.42E+09	4.69E+07	2.47E+09
20	20	0	2.69E+09	2.27E+07	2.72E+09
21	21	7.59E+08	2.59E+09	2.14E+07	3.37E+09
22	22	2.99E+09	2.47E+09	2.09E+07	5.48E+09
23	23	7.59E+08	2.75E+09	1.41E+06	3.51E+09
24	24	7.59E+08	2.73E+09	0	3.49E+09
25	25	7.59E+08	2.64E+09	1.29E+06	3.40E+09
26	26	7.59E+08	2.60E+09	0	3.36E+09
27	27	7.59E+08	2.49E+09	0	3.25E+09
28	28	7.59E+08	2.14E+09	674544	2.90E+09
29	29	0	1.14E+09	483636	1.14E+09
30	30	7.57E+08	0	0	7.57E+08
31	31	0	9.60E+08	0	9.60E+08
32	32	7.59E+08	2.32E+09	0	3.08E+09
33	33	0	2.17E+09	0	2.17E+09
34	34	0	1.85E+09	0	1.85E+09
35	35	7.59E+08	8.16E+08	0	1.57E+09
36	36	0	8.61E+08	0	8.61E+08
37	37	0	1.98E+09	0	1.98E+09
38	38	7.59E+08	2.07E+09	0	2.83E+09
39	39	0	2.64E+09	0	2.64E+09
40	40	7.59E+08	2.53E+09	0	3.29E+09
41	41	0	1.53E+09	0	1.53E+09
42	42	7.59E+08	2.07E+09	0	2.83E+09
43	43	0	1.85E+09	0	1.85E+09
44	44	7.53E+08	1.43E+09	0	2.18E+09
45	45	0	2.24E+09	0	2.24E+09
46	46	7.43E+08	2.12E+09	0	2.87E+09
47	48	0	2.54E+09	0	2.54E+09
48	49	0	2.70E+09	0	2.70E+09
49	50	0	9.63E+08	0	9.63E+08
50	51	7.43E+08	0	0	7.43E+08
51	52	7.58E+08	1.26E+09	0	2.02E+09
52	53	0	2.37E+09	0	2.37E+09
53	54	7.57E+08	1.04E+09	0	1.79E+09
54	55	7.59E+08	1.23E+09	0	1.99E+09
55	56	7.54E+08	0	0	7.54E+08
56	60	0	7.74E+08	0	7.74E+08
57	62	0	6.08E+08	0	6.08E+08
58	63	0	9.23E+08	0	9.23E+08
59	65	7.59E+08	0	0	7.59E+08
60	66	6.89E+08	0	0	6.89E+08
61	68	0	9.63E+08	0	9.63E+08
62	69	7.59E+08	0	0	7.59E+08
63	70	7.59E+08	0	0	7.59E+08
64	71	0	9.56E+08	0	9.56E+08
65	79	0	4.47E+08	0	4.47E+08
66	83	1.67E+08	0	0	1.67E+08

Flood Damage in Euro (Case study no 2; Option 1a)					
		Entire catchment			
No. of failed scenario's	SOBEK Case No	Damage in Euro in Southern polder	Damage in Euro in Northern polder	Damage in Euro in Emergency Retention polder	Damage in Euro in Entire catchment
Total Damage in €		4.25E+10	1.45E+11	1.07E+09	1.88E+11
No. Scenario's failed		39	58	26	66
Flood risks in 10^6 Euro (Case study no 2; Option 1a)					
		Entire catchment			
Number of scenario's		Southern polder	Northern polder	Emergency retention polder	Entire catchment
3000		14.18	48.26	0.36	62.80

Table E.6 Case study no 2, Option 1b (fixed weir), Flood damage per scenario in Euro and Flood risk in 10^6 Euro on basis of 3000 scenario's, considering the overtopping failure mechanism only and probability density functions for model parameters as given in Table 4.4, except for model parameters "Critical water depth for piping" and "Critical duration for exceedance of piping depth". Flood risks considering the entire catchment are given. Only scenario's in which damage occurred are given.

Flood Damage in Euro (Case study no 2; Option 1b)					
		Entire catchment			
No. of failed scenario's	SOBEK Case No	Damage in Euro in Southern polder	Damage in Euro in Northern polder	Damage in Euro in Emergency Retention polder	Damage in Euro in Entire catchment
1	1	3.32E+09	1.18E+10	7.72E+07	1.52E+10
2	2	3.25E+09	0	7.37E+07	3.32E+09
3	3	8.33E+07	5.49E+09	7.34E+07	5.65E+09
4	4	8.41E+07	5.42E+09	7.34E+07	5.58E+09
5	5	0	5.42E+09	6.80E+07	5.49E+09
6	6	8.32E+07	2.11E+09	7.34E+07	2.26E+09
7	7	0	2.61E+09	6.99E+07	2.68E+09
8	8	0	1.58E+09	7.33E+07	1.65E+09
9	9	0	1.66E+09	6.35E+07	1.72E+09
10	10	0	0	6.69E+07	6.69E+07
11	11	0	2.85E+09	5.28E+07	2.90E+09
12	12	0	2.29E+09	6.23E+07	2.36E+09
13	13	0	1.79E+09	5.73E+07	1.85E+09
14	14	0	1.22E+09	5.59E+07	1.28E+09
15	15	0	5.42E+09	4.73E+06	5.43E+09
16	16	7.59E+08	0	5.39E+07	8.13E+08
17	17	2.06E+09	2.70E+09	2.04E+07	4.78E+09
18	18	0	4.11E+09	2.07E+06	4.11E+09
19	19	0	2.36E+09	4.82E+07	2.41E+09
20	20	0	2.65E+09	2.41E+07	2.67E+09
21	21	7.59E+08	0	4.51E+07	8.04E+08
22	22	0	1.47E+09	4.50E+07	1.51E+09
23	23	0	2.62E+09	2.19E+07	2.64E+09
24	24	0	1.48E+09	2.56E+07	1.51E+09
25	25	0	0	2.83E+07	2.83E+07
26	26	0	2.42E+09	2.09E+07	2.44E+09
27	27	0	2.51E+09	4.79E+06	2.51E+09

Flood Damage in Euro (Case study no 2; Option 1b)					
		Entire catchment			
No. of failed scenario's	SOBEK Case No	Damage in Euro in Southern polder	Damage in Euro in Northern polder	Damage in Euro in Emergency Retention polder	Damage in Euro in Entire catchment
28	28	0	1.21E+09	4.58E+06	1.21E+09
29	29	0	1.05E+09	1.74E+06	1.05E+09
30	33	0	1.83E+09	0	1.83E+09
31	36	0	8.61E+08	0	8.61E+08
Total Damage in €		1.04E+10	7.69E+10	1.29E+09	8.86E+10
No. Scenario's failed		8	26	29	31
Flood risks in 10^6 Euro (Case study no 2; Option 1b)					
		Entire catchment			
Number of scenario's		Southern polder	Northern polder	Emergency retention polder	Entire catchment
3000		3.46	25.63	0.43	29.53

Table E.7 Case study no 2, Option 2a (adjustable weir), Flood damage per scenario in Euro and Flood risk in 10^6 Euro on basis of 3000 scenario's, considering the overtopping and piping failure mechanism and probability density functions for model parameters as given in Table 4.4. Flood risks considering the entire catchment are given. Only scenario's in which damage occurred are given.

Flood Damage in Euro (Case study no 2; Option 2a)					
		Entire catchment			
No. of failed scenario's	SOBEK Case No	Damage in Euro in Southern polder	Damage in Euro in Northern polder	Damage in Euro in Emergency Retention polder	Damage in Euro in Entire catchment
1	1	3.32E+09	1.18E+10	7.72E+07	1.52E+10
2	2	3.32E+09	4.46E+09	7.43E+07	7.85E+09
3	3	2.99E+09	5.49E+09	7.37E+07	8.56E+09
4	4	2.99E+09	5.43E+09	7.37E+07	8.49E+09
5	5	2.99E+09	5.30E+09	7.37E+07	8.37E+09
6	6	7.58E+08	4.08E+09	7.37E+07	4.91E+09
7	7	2.99E+09	2.60E+09	7.17E+07	5.67E+09
8	8	0	4.80E+09	7.17E+07	4.87E+09
9	9	7.59E+08	2.79E+09	6.39E+07	3.61E+09
10	10	7.59E+08	2.72E+09	6.37E+07	3.54E+09
11	11	7.59E+08	2.84E+09	6.23E+07	3.66E+09
12	12	0	2.49E+09	6.23E+07	2.55E+09
13	13	2.06E+09	2.22E+09	6.23E+07	4.34E+09
14	14	2.80E+09	1.19E+09	6.23E+07	4.05E+09
15	15	0	5.42E+09	6.23E+07	5.49E+09
16	16	7.59E+08	1.59E+09	6.12E+07	2.41E+09
17	17	7.59E+08	2.70E+09	5.30E+07	3.51E+09
18	18	7.59E+08	3.16E+09	5.13E+07	3.97E+09
19	19	0	2.38E+09	5.05E+07	2.43E+09
20	20	0	2.68E+09	4.91E+07	2.73E+09
21	21	7.59E+08	2.59E+09	4.63E+07	3.39E+09
22	22	2.99E+09	2.47E+09	4.59E+07	5.51E+09
23	23	7.59E+08	2.74E+09	3.76E+07	3.53E+09
24	24	7.59E+08	2.73E+09	2.55E+07	3.52E+09
25	25	7.59E+08	2.64E+09	1.98E+07	3.42E+09

26	26	7.59E+08	2.59E+09	1.28E+07	3.36E+09
27	27	7.59E+08	2.49E+09	3.14E+06	3.25E+09
28	28	7.59E+08	2.14E+09	0	2.90E+09
29	29	0	1.14E+09	0	1.14E+09
30	30	7.57E+08	0	0	7.57E+08
31	31	0	9.60E+08	0	9.60E+08
32	32	7.59E+08	2.32E+09	0	3.08E+09
33	33	0	2.17E+09	0	2.17E+09
34	34	0	1.85E+09	0	1.85E+09
35	35	7.59E+08	8.16E+08	0	1.57E+09
36	36	0	8.60E+08	0	8.60E+08
37	37	0	1.98E+09	0	1.98E+09
38	38	7.59E+08	2.07E+09	0	2.82E+09
39	39	0	2.64E+09	0	2.64E+09
40	40	7.59E+08	2.53E+09	0	3.29E+09
41	41	0	1.53E+09	0	1.53E+09
42	42	7.59E+08	2.07E+09	0	2.83E+09
43	43	0	1.85E+09	0	1.85E+09
44	44	7.53E+08	1.43E+09	0	2.18E+09
45	45	0	2.24E+09	0	2.24E+09
46	46	7.43E+08	2.12E+09	0	2.86E+09
47	48	0	2.54E+09	0	2.54E+09
48	49	0	2.70E+09	0	2.70E+09
49	50	0	9.63E+08	0	9.63E+08
50	51	7.43E+08	0	0	7.43E+08
51	52	7.58E+08	1.26E+09	0	2.02E+09
52	53	0	2.37E+09	0	2.37E+09
53	54	7.57E+08	1.04E+09	0	1.79E+09
54	55	7.59E+08	1.23E+09	0	1.99E+09
55	56	7.54E+08	0	0	7.54E+08
56	60	0	7.74E+08	0	7.74E+08
57	62	0	5.34E+08	0	5.34E+08
58	63	0	9.23E+08	0	9.23E+08
59	65	7.59E+08	0	0	7.59E+08
60	66	6.89E+08	0	0	6.89E+08
61	68	0	7.78E+08	0	7.78E+08
62	69	7.59E+08	0	0	7.59E+08
63	70	7.59E+08	0	0	7.59E+08
64	71	0	9.56E+08	0	9.56E+08
65	79	0	4.47E+08	0	4.47E+08
66	83	1.67E+08	0	0	1.67E+08
Total Damage in €		5.00E+10	1.44E+11	1.48E+09	1.95E+11
No. Scenario's failed		41	58	27	66

Flood risks in 10^6 Euro (Case study no 2; Option 2a)				
	Entire catchment			
Number of scenario's	Southern polder	Northern polder	Emergency retention polder	Entire catchment
3000	16.67	47.85	0.50	65.03

Table E.8 Case study no 2, Option 2b (adjustable weir), Flood damage per scenario in Euro and Flood risk in 10^6 Euro on basis of 3000 scenario's, considering the overtopping failure mechanism only and probability density functions for model parameters as given in Table 4.4, except for model parameters "Critical water depth for piping" and "Critical duration for exceedance of piping depth". Flood risks considering the entire catchment are given. Only scenario's in which damage occurred are given.

Flood Damage in Euro (Case study no 2; Option 2b)					
		Entire catchment			
No. of failed scenario's	SOBEK Case No	Damage in Euro in Southern polder	Damage in Euro in Northern polder	Damage in Euro in Emergency Retention polder	Damage in Euro in Entire catchment
1	1	3.32E+09	1.18E+10	7.72E+07	1.52E+10
2	2	3.32E+09	1.29E+09	7.43E+07	4.68E+09
3	3	2.99E+09	5.43E+09	7.43E+07	8.49E+09
4	4	2.99E+09	4.43E+09	7.43E+07	7.50E+09
5	5	7.59E+08	4.76E+09	7.37E+07	5.60E+09
6	6	7.59E+08	2.22E+09	7.37E+07	3.05E+09
7	7	0	5.30E+09	7.17E+07	5.37E+09
8	8	0	0	7.15E+07	7.15E+07
9	9	0	2.65E+09	6.39E+07	2.72E+09
10	10	0	2.37E+09	6.37E+07	2.44E+09
11	11	0	2.84E+09	6.23E+07	2.90E+09
12	12	2.99E+09	2.37E+09	6.23E+07	5.43E+09
13	13	0	1.79E+09	6.23E+07	1.85E+09
14	14	0	1.22E+09	6.23E+07	1.29E+09
15	15	0	5.30E+09	6.23E+07	5.36E+09
16	16	7.59E+08	1.46E+09	6.04E+07	2.28E+09
17	17	1.19E+09	2.69E+09	5.28E+07	3.93E+09
18	18	0	3.48E+09	5.13E+07	3.53E+09
19	19	7.59E+08	2.26E+09	5.06E+07	3.06E+09
20	20	0	2.63E+09	4.91E+07	2.68E+09
21	21	7.59E+08	0	4.59E+07	8.05E+08
22	22	0	1.47E+09	4.55E+07	1.51E+09
23	23	0	2.61E+09	3.65E+07	2.65E+09
24	24	0	1.48E+09	2.40E+07	1.51E+09
25	25	0	0	1.57E+07	1.57E+07
26	26	0	2.42E+09	1.10E+07	2.43E+09
27	27	0	2.51E+09	2.49E+06	2.51E+09
28	28	7.53E+08	1.17E+09	0	1.92E+09
29	29	0	1.05E+09	0	1.05E+09
30	33	0	1.83E+09	0	1.83E+09
31	36	0	8.60E+08	0	8.60E+08
Total Damage in €		2.13E+10	8.17E+10	1.48E+09	1.05E+11
No. Scenario's failed		12	28	27	31
Flood risks in 10^6 Euro (Case study no 2; Option 2b)					
		Entire catchment			
Number of scenario's		Southern polder	Northern polder	Emergency retention polder	Entire catchment
3000		7.11	27.21	0.49	34.82

F Preposterior Bayesian Analysis

Consider a decision-making problem involving some unknown or uncertain parameter a. This parameter could be a deterministic value, the mean of a distribution, a standard deviation, a probability or whatever else. Assign to the unknown parameter a probability density function $f'(a)$. As long as no specific information is available, this distribution will be very broad. After gathering more information it will become sharper, its limit being a spike function, indicating that the value of the parameter has been fully identified.

Consider a situation where an optimal value for a design parameter x has to be found. In flood engineering x could be, for instance, the dike height. Assume that for a given value of the unknown parameter a the buildings costs C_o and the failure probability P_f are known. In that case we will look for the value of x that minimizes the total costs given by:

$$C(a, x) = C_o(x) + P_f\,(a, x)\,D \qquad \text{(F.1)}$$

where:
$C_o(x)$ = the direct costs of design choice x
$P_f(x,a)$ = probability of failure
D = damage occurring due to failure

As a, however, is a random variable, we can only consider the minimization of the expectation E′{.} of $C(a, x)$:

$$C'_{opt} = \min \mathrm{E}'\{C(a,x)\} = \min \int C(a,x)\, f'(a).\mathrm{d}a \qquad \text{(F.2)}$$

The optimal value is called C'_{opt} . Assume that the uncertainty in the parameter a can be reduced by carrying out some kind of experiment, observation or calculation. Such additional information will probably improve the decision and save money. On the other hand, it will cost money to gather the information. The decision to make the observation will be taken only if the expectation of the new total cost plus the cost of getting the additional information is less then the present cost expectation:

$$\mathrm{E}(C''_{opt}) + C_{exp} < C'_{opt}\,, \qquad \text{(F.3)}$$

where C_{exp} represents the costs of “carrying out the experiment or the calculation”. The double prime is a notation for the distributions and expectations after the additional observation; the single prime indicates the situation before the additional experiment.

We need now to evaluate $\mathrm{E}(C''_{opt})$. Let the (inaccurate) observation be the value a*. The posterior probability distribution function $f''(a|a^*)$, which can be determined from Bayes theorem, is given by:

$$f''(a|\, a^*) = c\, P(\text{observation } = a^* \mid a\,)\, f'(a) \qquad \text{(F.4)}$$

The probability P(observation = a* | a) contains a model for the inaccuracy of the observation given the true value a. The symbol c stands for a normalization constant which ensures that the area under the graph of $f''(a)$ is equal to 1.0. An adjusted expectation value of $C(a,x)$ is then given by:

$$C''_{opt} = \min E''\{C(a,x)\} = \min \int C(a,x)\, f''(a \mid a^*).da \qquad \text{(F.5)}$$

However, since the observation a* is unknown before it is made, it is only possible to calculate the expected value of this minimum:

$$E'(C''_{opt}) = E' \min E''\{C(a,x)\} = \int [\min \int C(a,x)\, f''(a \mid a^*).da\ ]\, f'(a^*).da^* \qquad \text{(F.6)}$$

This completes all ingredients necessary to take the decision to make an additional "observation".

As an example consider the following cost function (see Table F.1 for descriptions and data):

$$C = C_o + c\,R + D\,P(Z<0) \qquad \text{(F.7)}$$

The limit state function Z is given by:

$$Z = R - S \qquad \text{(F.8)}$$

Let R be a deterministic resistance and let S be a random Gaussian load with unknown mean and a known standard distribution. For the unknown mean we have done 2 observations S1 and S2 starting from completely uninformative prior information) and the question is whether it makes sense to do a third one. The cost of this observation is 10 Euro.

Table F.1 Data for Preposterior Bayesian Analysis

Co	Initial cost	1000	Euro
c	Cost per unit resistance	100	Euro / kN
D	Cost of damage	100000	Euro
C_{obs}	Cost of additional observation of S	10	Euro
R	Resistance	Variable	kN
$\sigma(S)$	Standard deviation of S	2	kN
S1	First observation for the load S	5	kN
S2	Second observation for the load S	7	kN

Given the information after n=2 observations of S, the load has a normal distribution with a mean of (S1+S2)/n = 6 kN and a standard deviation of $\sigma(S)\sqrt{(1+1/n)} = 2\sqrt{(1+1/2)} = 2.5$ kN. The optimal values are:

R = 14 kN
P(F) = 0.0007
$C = C_{opt} = 1000 + 14*100 + 0.0007*100000 = 2470$ Euro

For the next observation we first have the costs C_{obs}. The optimum costs given a third observation S3 can be formulated as follows:

$$C'' = C_{obs} + \min (Co + c\,R + D\,P(R-S<0 \mid S3)) \qquad \text{(F.9)}$$

The distribution of S given S3 is normal with mean (S1+S2+S3)/3 and a standard deviation equal to 2 √(1+1/3). For every possible value of S3 we now optimize C" in the same way as we did in the case of n=2 observations. So we will get a function $C''_{opt}(S3)$ (see Figure F.1). The expected value of this optimum is:

$$E(C''_{opt}) = \int C''_{opt}(S3)\, f(S3)\, dS3 \qquad \text{(F.10)}$$

Where f(S3) is a normal probability density function with mean 6 kN and standard deviation 2.5 kN. The result is E(C"opt) = 2250 Euro. As this is lower then the 2470 Euro of the original design it makes sense to do the third observation. If doing the observation would cost over 230 Euro, it would be more economical to skip a third observation.

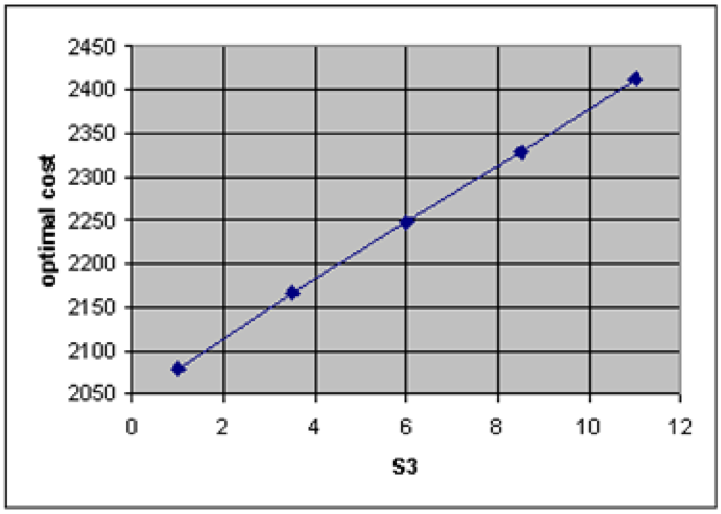

Fig F.1 Total cost as function of observation S3

Application in a Monte Carlo simulation

In the case of a Monte Carlo simulation, the parameter *a* represents the failure probability P_f. After N runs (with N sufficiently large), the prior of P_f is represented by a normal distribution with mean p and standard deviation $\sqrt{(p/N)}$. This can be written as:

$$P_f = p + u\sqrt{(p/N)} \qquad \text{(F.11)}$$

Here, *u* is a random variable with a standard normal distribution. The costs for a given value of *p* and *x* are:

$$C(p,x) = C_o(x) + P_f(x)\,S = C_o(x) + S\,\{p(x) + u\sqrt{(p(x)/N)}\} \qquad \text{(F.12)}$$

(It is assumed that in some way or another it is known how *p* and *x* are related to each other. This can be done via an extensive Monte Carlo analysis, but also using an analytical expression). The value of *x* for which the expected value of *C(p,x)* must now be sought:

$$C'_{opt} = \min E\,\{\,C_o(x) + S\,\{p(x) + u\sqrt{(p(x)/N)}\}\} = \min\,\{C_o(x) + S\,p(x)\} \qquad \text{(F.13)}$$

The variation therefore does not play a role. Using the above expression, it is possible to evaluate x and the corresponding costs.

In a series of extra Monte Carlo simulations $p(x)$ will change in a random way to $p(x)+vs$ with s a standard deviation (see (F.15 to F.17) and v a variable with normal distribution. Also the standard deviation of $p(x)$ will change, but this does not matter. The expected value of the minimum that corresponds with the new failure probability is given by:

$$E\,\{C''_{opt}\} = E \min \{ C_o(x) + S\{ p(x) + v\,s \} \} \qquad \text{(F.14)}$$

Because the operator for the expected value comes before the minimum operator, it is not possible to eliminate the variable v as in the expression for the evaluation of C'_{opt}.

For completeness sake, the value of s will be evaluated. After N drawings, the failure probability has a normal distribution with mean p and standard deviation $\sqrt{(p/N)}$. After ΔN extra observations, the number of failure events will increase from Np to

$$Np + \Delta Np + u\ \sqrt{(p/N)}\sqrt{\Delta N}. \qquad \text{(F.15)}$$

The failure probability after ΔN more draws is obtained by dividing by $N+\Delta N$:

$$p(N+\Delta N) = p + u\ \sqrt{(p\Delta N/N)}./\,(N+\Delta N) \qquad \text{(F.16)}$$

or:

$$s = \sqrt{(p\Delta N/N)}\,/\,(N+\Delta N) \qquad \text{(F.17)}$$

Note that it is not useful to carry out a small extra number of runs (the number of runs will be too small to possibly change the decision). The only options are either not to carry out any extra runs or to make many more runs. It is possible to build a strategy, for example by carrying out 1000 runs and then, having obtained a certain result, to decide whether to continue.