Probabilistic risk assessment of a system of dike ring areas

Main Report

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PREFACE

This is the report of my master's thesis entitled: ‘Probabilistic risk assessment of a system of dike ring areas.’ The report consists of two documents: the ‘Main report’ and the ‘Appendices’.

This thesis marks the end of my studies Civil Engineering at the University of Technology Delft and has been executed at the ‘Road and Hydraulic Engineering Division of the Directorate General of Public Works and Water Management’ and ‘TNO Building and Construction Research’.

I would like to thank the members of the committee for their interest and constructive criticism during the realization of this project.

W.D. van der Wiel
November 2003
SUMMARY

Background and motive
Large parts of The Netherlands are situated below mean sea level or below the high water levels of the major rivers. Without measures of protection these areas would regularly be inundated. Therefore large water defence structures along the coast and rivers have been built over the centuries. These structures steadily developed into closed rings of water defences, the so-called 'dike ring areas'. The flood prone areas in Netherlands are currently subdivided into 53 of these dike ring areas, which are still the basis of the current flood defence policy (appendix A).

Currently large efforts are made in order to determine the probabilities of and risks of flooding (risk = probability of flooding x consequences) of these 53 dike ring areas) in order to provide a picture of the actual safety of the dike ring areas. In these calculations however, each dike ring area is assessed completely separately from the rest of the system (i.e. other dike ring areas and rivers). It is expected however, that the protection level of one dike ring area depends on the protection level of other dike ring areas; a certain hydraulic interaction between dike ring areas exists. This interaction will probably influence the calculated probabilities and risks of flooding of these dike ring areas. These hydraulic interactions are also important in the assessment of future measures, such as dike improvement and the application of emergency storage areas.

A distinction is made between an 'active' and a 'passive' approach of these interactions between dike ring areas. A passive approach means, that the (currently unknown) effects of the interaction between the dike ring areas are silently accepted: nature/chance decides what will happen when a flood occurs. In an active approach, measures are taken to try to use the interaction between dike ring areas when floods are imminent in order to reduce the expected damage: the government/society tries to decide what will happen in case of a flood (e.g. the application of an emergency storage area).

Objective and method of approach
The objective of this thesis is to acquire insight in the quantitative effects of these interactions on the probabilities and risks of flooding of dike ring areas. Therefore three small system configurations are selected on the basis of the Dutch upper river system (displayed schematized in Figure 0.1). Each system configuration represents different ways of interaction (a distinction is made between positive and negative interaction):

I. Interaction through river (failure of A/B might prevent failure of B/A: positive)
II. Shortcut between dike ring areas (failure of A might induce failure of B: negative)
III. Shortcut between rivers (failure of A might induce failure of B: negative);

Figure 0.1 Three system configurations, white dots are possible points of failure

The probabilities and risks of flooding of these dike ring areas are calculated by means of Monte Carlo Simulations. For that purpose a model has been developed which provides a simplified approach of the hydraulic systems shown above. In the calculations the safety of the dike ring areas is determined including and without these interactions.
different ways of interaction, in order to enable comparison between the two approaches: isolated (omits interaction) and the integrated (includes interaction) risk assessment.

**Results**

Figure 0.2 shows the results of the calculations of system configuration I. On the horizontal axis the damage inflicted to the system is displayed and on the vertical axis the probability of exceedance of that damage (top) or the probability density (bottom). It is to be seen that the assessment without interaction is a conservative approach, as it overestimates the probabilities and risks of flooding of the total system and of the single dike ring areas A and B. However, if negative interaction plays a role as well, the assessment without interaction is no longer by definition a conservative approach, as it might underestimate the probabilities and risks of flooding.

![Figure 0.2 Results system configuration I (top) and II (bottom)](image)

**Conclusions**

Despite the fact that the system configurations (Figure 0.1) are highly schematized and simplified compared to a realistic river system, the calculations offer insight in the effects of the interaction between dike ring areas on the risk of these dike ring areas. Insight in the effects of local dike improvement on the system is offered as well. The probabilities and risks of flooding of dike ring areas that are calculated including the interaction can differ considerably from the (currently applied) isolated assessment (without interaction): calculated probabilities and risks of flooding can reduce (compared to the isolated assessment), but also increase, depending on the system configuration. Local dike improvement always decreases the probability of flooding of the whole system, although effects can turn out negative locally. However, local dike improvement might increase the risk of the system: a risk reduction of a dike ring area can be overcompensated by an increase of the risk of another dike ring area.

It is therefore concluded that for a correct assessment of the current probabilities and risks of flooding of dike ring areas along the rivers as well as for a correct cost-benefit analyses of possible future measures (dike improvement, emergency storage areas) an assessment which includes the interaction between dike ring areas is required.
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<tr>
<td>A</td>
<td>Area of cross-section of flow in a river [m²]</td>
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</tr>
<tr>
<td>Barea</td>
<td>Width of dike ring area [km]</td>
<td></td>
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</tr>
<tr>
<td>c</td>
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<td></td>
</tr>
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<td>cf</td>
<td>Roughness</td>
<td></td>
</tr>
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<td>CHW</td>
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<td></td>
</tr>
<tr>
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</tr>
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</tr>
<tr>
<td>D</td>
<td>Damage [€]</td>
<td></td>
</tr>
<tr>
<td>Dmax</td>
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<td></td>
</tr>
<tr>
<td>e</td>
<td>Parameter used in different formulas (values from context)</td>
<td></td>
</tr>
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<td></td>
</tr>
<tr>
<td>f(x)</td>
<td>Probability density function of x</td>
<td></td>
</tr>
<tr>
<td>Fr</td>
<td>Froude number</td>
<td></td>
</tr>
<tr>
<td>g</td>
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<td></td>
</tr>
<tr>
<td>Hcr,overflow</td>
<td>Critical dike height regarding overflow [m]</td>
<td></td>
</tr>
<tr>
<td>Hcr,piping</td>
<td>Critical dike height regarding piping [m]</td>
<td></td>
</tr>
<tr>
<td>Hriver</td>
<td>Water level in river [m]</td>
<td></td>
</tr>
<tr>
<td>i</td>
<td>River inclination [-]</td>
<td></td>
</tr>
<tr>
<td>I</td>
<td>Variable construction costs of a dike [€/m]</td>
<td></td>
</tr>
<tr>
<td>I₀</td>
<td>Initial costs of a dike [€]</td>
<td></td>
</tr>
<tr>
<td>Iₜ</td>
<td>Total construction costs of a dike [€]</td>
<td></td>
</tr>
<tr>
<td>k</td>
<td>Roughness coefficient of Nikuradse [m]</td>
<td></td>
</tr>
<tr>
<td>L</td>
<td>Length of dike ring area [km]</td>
<td></td>
</tr>
<tr>
<td>ln()</td>
<td>natural logarithm</td>
<td></td>
</tr>
<tr>
<td>m</td>
<td>Roughness coefficient of Manning [m¹/³]</td>
<td></td>
</tr>
<tr>
<td>N</td>
<td>Number of runs in a Monte Carlo Simulation</td>
<td></td>
</tr>
<tr>
<td>p</td>
<td>Number of runs in a Monte Carlo Simulation in which failure occurs</td>
<td></td>
</tr>
<tr>
<td>Pf</td>
<td>Probability of failure [1/year]</td>
<td></td>
</tr>
<tr>
<td>Q</td>
<td>River discharge [m³/s]</td>
<td></td>
</tr>
<tr>
<td>Q*</td>
<td>Support variable, used in calculations of volume of inundation [m³/s]</td>
<td></td>
</tr>
<tr>
<td>Qav</td>
<td>Average river discharge during times when no discharge wave occurs [m³/s]</td>
<td></td>
</tr>
<tr>
<td>Qbreach(t)</td>
<td>The discharge through the breach as a function of time [m³/s]</td>
<td></td>
</tr>
<tr>
<td>Qcr,esa</td>
<td>Critical discharge regarding emergency storage area [m³/s]</td>
<td></td>
</tr>
<tr>
<td>Qfailure</td>
<td>River discharge at the moment of failure [m³/s]</td>
<td></td>
</tr>
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<td>Qmax</td>
<td>Discharge in top of a discharge wave [m³/s]</td>
<td></td>
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<td>Qmax,design</td>
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<td></td>
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<tr>
<td>Qu</td>
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<td></td>
</tr>
<tr>
<td>R</td>
<td>Hydraulic radius [m]</td>
<td></td>
</tr>
<tr>
<td>R</td>
<td>Risk [€/Year]</td>
<td></td>
</tr>
<tr>
<td>t₀</td>
<td>The moment that for the first time Hriver &gt; Hcr,piping [s]</td>
<td></td>
</tr>
</tbody>
</table>
List of symbols

\( T_{cr,piping} \)  Critical time of exceedance (in combination with \( H_{cr,piping} \)) regarding piping [h]

\( T_{end} \)  The time interval in which \( B_{\text{max}} \) is reached [h]

\( U \)  Flow velocity [m/s]

\( V_{\text{cap}} \)  Storage volume of dike ring area [m\(^3\)]

\( V_{in} \)  Volume of inundation [m\(^3\)]

\( w \)  River width [m]

\( x \)  Distance along the river [km]

\( Z \)  Reliability function

\( \alpha \)  Parameter used in calculation of volume of inundation

\( \beta \)  Support variable, used in calculations of volume of inundation

\( \delta_{\text{discharge}} \)  Discharge factor

\( \Delta Q_{\text{max}} \)  Lowering of the top of the discharge wave (\( Q_{\text{max}} \)) during propagation [m\(^3\)/s]

\( \Delta Z(t) \)  Momentary difference between the momentary river water level and the land level in the dike ring area [m]

\( \varepsilon \)  Relative deviation

\( \mu \)  Contraction coefficient in formula for \( Q_{\text{breach}(t)} \) (value = 1)

\( \mu_x \)  Mean of stochastic variable \( x \)

\( \sigma_x \)  Standard deviation of stochastic variable \( x \)
1 INTRODUCTION

Large parts of The Netherlands are situated below mean sea level or below the water levels of the major rivers. Without measures of protection these parts would regularly be inundated. Therefore large water defence structures along the coast and rivers have been built over the centuries. These structures steadily developed into closed rings of water defences, the so-called ‘dike ring areas’. The flood prone areas in The Netherlands are currently subdivided into 53 dike ring areas, which are still the basis of the current flood defence policy (appendix A).

At the moment large efforts are made in order to determine the probabilities and risks of flooding of these 53 dike ring areas. The results of these calculations will provide a picture of the current protection level of the defence structures and will be a contribution to a political and societal discussion about the required protection level of the Dutch water defence structure.

In these calculations each dike ring area is approached completely separately from the rest of the system. It is expected however, that the protection level of one dike ring area depends on the protection level of other dike ring areas; a certain hydraulic interaction between dike ring areas exists. This interaction will probably influence the calculated probabilities and risks of flooding of these dike ring areas. These hydraulic interactions are also important in the assessment of future measures, such as dike improvement and the application of emergency storage areas.

The objective of the research project is to acquire insight in the quantitative effects of this interactions between dike ring areas on the probabilities and risks of flooding of these dike ring areas.

The structure of this report is as follows (see also Figure 1.1): chapter 2 describes the background of the levels of protection of the water defence structures in the Netherlands. The historical development, the current situation as well as the expected future measures (such as the application of emergency storage areas) will be discussed. In chapter 3 the problem analysis is presented. Different ways in which interaction between dike ring areas can occur in a river system are described. This analysis leads to the problem definition and objective of this research project: three important system configurations are selected and will be analysed in the following chapters. For this purpose a model is developed which is described in chapter 4. In chapter 5 the various system configurations are discussed in more detail. In chapter 6 the results (probabilities and risks of flooding) are presented and analysed. Chapter 7 closes with conclusions and recommendations.

![Figure 1.1 Structure of the report](image-url)
In this chapter important aspects regarding water defences and levels of protection are discussed. It starts with an explanation of the indispensability of water retaining structures in The Netherlands (paragraph 2.1), followed by the discussion regarding the determination of the level of protection of these structures in paragraph 2.2.

2.1 **Necessity of Water Defences**

Until the middle ages people in the Netherlands mainly lived on the higher parts of the land and on artificial hills. Consequently the regular floods from rivers and sea did hardly cause any damage to their houses and farms. As the population increased, lower parts of the land were made suitable for agriculture by constructing a drainage system (±1000 AD). This resulted in a fall of the land level and flooding started to become a problem. By this time (±1200 AD) the first dikes were constructed. The building efforts began locally but soon it was discovered that the best way to protect an area was to build a closed ring of water defences around it. These so called ‘dike ring areas’ are still the basis of the current flood defence policy of The Netherlands (Appendix A).

Enhanced drainage techniques, at first windmills (±1400 AD), later mechanical pumping (±1850 AD), were the cause of a further fall in land level (see also Figure 2.1).

The main consequence of this development is that currently 26 % of The Netherlands is situated below N.A.P. (3 meters on average with a minimum of even 6.7 meters below N.A.P.) In addition, parts of The Netherlands that are located along the major rivers are several meters below the river water level as well, especially during times of high river discharges. Without an extensive system of water defences consisting of dikes, dunes and storm surge barriers (Appendix B), about 2/3 of the Netherlands would regularly be inundated by sea or by river (Figure 2.2). Over the last hundreds of years these flood prone areas have become safer, but also more valuable. Considering the current economical and social values in these areas it becomes clear that nowadays the consequences of flooding will be very serious.
2.2 PHILOSOPHY OF PROTECTION

It is important to understand some historical events and developments regarding the philosophy of protection of water defences. These can be found in paragraph 2.2.1. The current developments are discussed in paragraph 2.2.2. The future measures are described in paragraph 2.2.3.

2.2.1 HISTORICAL DEVELOPMENT: PROBABILITIES OF EXCEEDANCE OF HIGH WATER LEVELS

Until the first half of the last century the height of flood defences was determined mostly by experience. The design height of a flood defence structure was defined as ‘the highest known water level on site’ plus a certain margin (usually about 1 m).

As a result of the ‘Delta Flooding Disaster’ in 1953 (large parts of the south-west of The Netherlands flooded and over 1800 people died4) the Dutch government established the Delta committee. This committee developed a new safety approach, in which statistical considerations regarding the frequency of occurrence of extreme water levels started to play a role: a scientific way was determined to define an economic optimal protection level. This should be done by an analysis of the risk of flooding (probability of flooding x consequences of flooding) and the costs of protection (Appendix C). However, at that time the probability of flooding could not be determined accurately enough due to lack of insight in the failure mechanisms of the different defence structures. In addition, lack of calculation capacity existed. So the Delta committee adopted a simplified approach, which contained a probability of exceedance of design high water levels on which the height of the water defences should be based. A probability of exceedance of 1/10,000 per year (corresponding with + 5 m N.A.P. at Hoek van Holland, the so-called ‘base level’) was assigned to Central and Northern Holland. For other parts along the coast higher probabilities of exceedance were applied varying from 1/4000 per year to approximately 1/1500 per year because of less severe consequences of flooding in those areas5.

In the following period probabilities of exceedance were determined for the water defences along the rivers as well. The historical (and future) development is shown in Table 2.1.

Table 2.1 Historical (and expected future) development probabilities of exceedance of defence structures and corresponding design discharges along the rivers

<table>
<thead>
<tr>
<th>Year</th>
<th>Body</th>
<th>Probabilities of exceedance</th>
<th>Discharge Rijn [m³/s]</th>
<th>Discharge Maas [m³/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1956</td>
<td>Dutch government</td>
<td>1/3000</td>
<td>18.000</td>
<td>-</td>
</tr>
<tr>
<td>1977</td>
<td>Becht Committee</td>
<td>1/1250</td>
<td>16.500</td>
<td>3.650</td>
</tr>
<tr>
<td>1992</td>
<td>Boertien Committee</td>
<td>1/1250</td>
<td>15.000</td>
<td>3.650</td>
</tr>
<tr>
<td>1996</td>
<td>HR19966 Delta Act Major Rivers</td>
<td>1/1250</td>
<td>15.000</td>
<td>3.650</td>
</tr>
<tr>
<td>2001</td>
<td>HR 2001 ‘Room for rivers’7</td>
<td>1/1250</td>
<td>16.000</td>
<td>3.800</td>
</tr>
<tr>
<td>2050</td>
<td>‘Room for rivers’</td>
<td>1/1250</td>
<td>18.000</td>
<td>4.600</td>
</tr>
</tbody>
</table>

The initial plans of 1956 have been changed several times. In 1977 the Becht committee determined less strict probability of exceedance because society protested violently

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5 The water level reached in 1953 appeared to have an exceedance frequency of about 1/300 per year (+3,85 m + N.A.P. at Hoek van Holland).
6 & 7 RIZA, Hydraulische randvoorwaarden 2001: maatgevende afvoeren Rijn en Maas, Arnhem, 2001; Hydraulic boundary conditions for all dike ring areas in The Netherlands formulated by the Dutch Government in the framework of the five-year verification of the water defence structures stated in the Flood Defence Act (explained next)
7 Explained in paragraph 2.2.3
against the execution of the works of improvement due to the damage that was inflicted on the landscape. In 1992 the Boertien committee decreased the design discharge again (while the exceedance frequency remained unchanged). In 1993 and 1995 The Netherlands were startled by high river discharges\(^8\), which caused dangerous situations. In 1995 certain areas even had to be evacuated (200,000 people, fortunately unnecessarily). In response the government adopted a new law [Major Rivers Delta Act, 1995], in which the immediate strengthening of the weakest parts of the defence structures along the rivers (about 146 km) is stated. Due to expected climate change the design river discharges in the future will change again (The way these changes are dealt with is discussed in paragraph 2.2.3).

### 2.2.2 Current Developments: Probabilistic Risk Assessment Single Dike Ring Area

In the Flood Defence Act, 1995 (appendix A), protection standards are assigned to the water defences surrounding a dike ring area and are expressed as an average probability of exceedance per year of the design high water level. These standards are based on the results of the mentioned committees and every single dike section\(^9\) has to be designed and verified according to these standards. The verification must take place every five years, in order to make sure that the defences still comply with the prescribed protection standards despite any changed conditions (paragraph 2.2.3).

However, it is currently unclear whether these protection standards themselves are still up to date. This is one of the reasons that in 1992 the TAW (Technical Advisory Committee on Water Defences) has started the research program ‘Flooding risks, a study of probabilities and consequences’. The main goal of this research program, following the initial plans of the Delta committee, is to enable a risk assessment (determination of probability of flooding and consequences) of a dike ring area. This calculation is intended for the following reasons:

- Together with the construction costs an economic optimization can be carried out which enables a cost-benefit-calculation of improvement measures (appendix C);
- Due to the increase of economical value and population over the years, the economic optimal probability of flooding might have changed since 1960;
- The flooding risk should be compared with risks in different fields: e.g. risks caused by transport or industry;
- The current design guides regarding dikes, dunes and special structures are not built up completely consistently from one fundament. The flooding risk approach pursues a more uniform approach in the future.

The mentioned TAW research project is roughly set up as follows:
1. Determination of the probability of flooding of a dike ring area;
2. Determination of the risk of flooding of a dike ring area;
3. Determination of the economic optimal level of protection.

---

\(^8\) The probability of exceedance of the discharges was about 1/100 per year (Rijn 12.060 m\(^3\)/s, Maas 3000 m\(^3\)/s).

\(^9\) Along one dike section the strength and load values can be assumed constant in space.
1 Probability of flooding of a dike ring area
The probability of flooding of a dike ring area differs from the probability of exceedance of a dike section (as used currently in the Flood Defence Act) for the following reasons:

> several failure modes are taken into account (instead of one: overflow/overtopping);
> uncertainties are taking into account beforehand systematically and controllably (instead of applying safety margins afterwards);
> the length of the dike ring plays a role: the longer the dike ring area, the higher the probability of flooding;
> simultaneous threats on one dike ring area are combined\(^\text{10}\)

The TAW [2000] demonstrated that it is now very well possible to estimate the probability of flooding of a dike ring area (for this purpose a computer program called PC RING has been developed, appendix D). This conversion of probability of exceedance to probability of flooding is also intended in the Flood Defence Act\(^\text{11}\).

2 Risk of flooding of a dike ring area
In this context risk is defined as the multiplication of the probability of flooding and the consequences of that flooding. These consequences (or damages) can vary from psychological and societal damage, environmental damage to economical damage and even casualties. A standard method for the determination of by inundation inflicted damage has been developed [Vrisou van Eck, 2000] in which the consequences are expressed in € (economical damage) and in number of casualties. This damage depends on: inundation depth, flow velocity and rising velocity of the inundating water. Yet research is still to be done on this subject. With these consequences of flooding known, the risk of flooding of a dike ring area can be determined (Figure 2.3). At the moment large efforts are made to determine these risks of flooding of all 53 dike ring areas in The Netherlands\(^\text{12}\). This will provide a picture of the level of protection and the risk of dike ring areas in The Netherlands.

3 Economic optimization
The economic optimal level of protection has to be determined. The costs of improvement measures will be compared to the achieved risk-reduction (Appendix C).

A broad discussion has to take place both in politics and in society about the required level of protection of water defences in The Netherlands. The research project of the TAW provides the objective technical considerations, but other more subjective political and social considerations play a role as well. Eventually the government has to decide

\(^{10}\) Consider for example: dike ring area 13 (appendix A) is threatened from both the North sea and the IJsselmeer. Assume: protection level north sea side = 1/10,000 and IJsselmeer side = 1/10,000. In case of independent threats, the protection level of dike ring area 13 becomes approximately 1/5000.

\(^{11}\) 'In accordance with and as replacement of the probability of exceedance, the protection standard for each dike ring area shall be determined by order in council as the average probability of flooding as a result of the breach of a primary flood defence.' (FDA, 1995)

\(^{12}\) These calculations are performed by a special body: V(eiligheid) N(ederland) in K(aart) which is a part of DWW, Rijkwaterstaat.
about the required level of protection, which might lead to different protection standards than are used at this moment (Appendix A).

2.2.3 FUTURE MEASURES: ROOM FOR RIVERS AND FLOOD STORAGE

It is expected that in the future climate changes will affect both the mean sea level (rise of ±60 cm) and the river discharges (Table 2.1 on page 4). Next to dike improvement, other kind of measures, which could be applied to meet the new requirements, are subject of research: ‘room for rivers’ and ‘flood storage’.

Room for rivers

The human interventions in the course of time confined the flow of the river water to a relatively small and rigid area. Sedimentation can only take place in this small area resulting in higher flood plains as the land behind the dikes is falling (Figure 2.1 on page 3). Higher dikes will create larger differences between river water level and land level and consequently larger volumes of inundation in case of flooding. In order to stop this process, the ‘Room for rivers’ policy line was adopted by the government containing strategic measures of which a few important ones are displayed in (Figure 2.4). Some of these measures are currently implemented in The Netherlands.

Flood storage

In the framework of ‘Room for rivers’ a lot of attention is given to the retention and especially the emergency storage areas: at times of high river discharges water is stored in special areas to lower the river water level downstream. A retention area is included in the determination of the design water level. Emergency storage areas however, are only brought into use when an uncontrolled flood is expected in order to cope with the residual risk. Recently the Luteijn committee concluded that emergency storage areas would be useful and necessary in The Netherlands and proposed some areas, which could be used as storage area to the government (Figure 2.5). A decision has not been taken yet. The TAW [Trouw, October 8, 2002] questions both the effectiveness of the storage areas and the considerations of the committee: a lot of uncertainties are not accounted for.

Figure 2.4 Possible measures in the framework of ‘Room for rivers’

Figure 2.5 Possible emergency storage areas in The Netherlands

13 Luteijn, D., Gecontroleerd overstromen, advies van de commissie noodoverloopgebieden, 2002
3 PROBLEM ANALYSIS

In this chapter the problem analysis of the thesis is deducted. It starts with a general introduction (paragraph 3.1), followed by an explanation of the two ways in which the safety of a river system can be approached (paragraph 3.3). Paragraph 3.2 describes and analyses the hydraulic interaction between dike ring areas, followed by the formal definition of the problem (paragraph 3.3) as well as the objective (paragraph 3.5).

3.1 INTRODUCTION

Currently a lot of attention is given to the safety of flood prone areas in The Netherlands. As described in the previous chapters, large efforts are made in order to calculate the actual probabilities and risks of flooding of the 53 dike ring areas. In addition, expected change of the climate on a worldwide scale compels to measures of improvement (paragraph 2.2.3: ‘Room for rivers’).

All the calculations concentrate on the determination of the actual probability and risk of flooding for each single dike ring area. This approach results from the Dutch legislation, in which safety standards are assigned to each single dike ring area (appendix A). As is also to be seen from this appendix, the dike ring areas in The Netherlands are part of a larger system of rivers and dike ring areas, which slopes to the west in its entirety. Due to this configuration a mutual dependence between the safety of dike ring areas exists, caused by different kinds of hydraulic interaction. Considering these interactions, it is not correct to analyse a dike ring area completely isolated from the rest of the system, which is actually done at the moment (paragraph 2.2.2). The different kinds of hydraulic interaction between dike ring areas are discussed in the next paragraph.

3.2 INTERACTION BETWEEN DIKE RING AREAS

3.2.1 WAYS OF INTERACTION

When a system of rivers and dike ring areas is considered (e.g. Figure 3.4 on page 12, the situation in The Netherlands), three important ways of interaction can be distinguished:
1. Interaction through a river;
2. Shortcut between dike ring areas;
3. Shortcut between rivers/river branches.

1 - Interaction through a river

Before a river system is discussed, first the situation of dike ring areas bordering a lake (with finite dimensions) is analysed. It is expected that the weakest dike ring area will fail first during uniform rise of the lake water level (e.g. due to wind). After failure the other dike ring areas might be saved due to the fall in the water level of the lake (this is only true if the volume of the lake is not too large compared to the drained volume).
In the situation of dike ring areas along a river, a time-aspect has to be added because of the propagation of the discharge wave: the first dike ring area (in time), which will not be able to cope with the discharge wave will fail, which is not necessarily the weakest, nor the most upstream dike ring area. Failure of one dike ring area might prevent failure of another dike ring area due to the fall of river water level (Figure 3.1). This is why retention and emergency storage areas are considered at the moment. Application of those areas is a deliberate (active) application of interaction through the river. It has to be noted that this way of interaction does not occur between dike ring areas along the coast: failure of a dike ring area does not result in a fall of sea level because of the ‘infinite’ area and volume of the sea.

Figure 3.1 Interaction through river: cross-section failure of A prevents failure of B

2 - Shortcut between dike ring areas
Two dike ring areas can be separated by a dike. A compartmentalizing dike is situated between two dike ring areas with the same protection standard (according to the Flood Defence Act, 1995), a separating dike is situated between two dike ring areas with different protection standards\(^{14}\). This way of interaction is especially applicable to dike ring areas along the upper rivers in The Netherlands because that area slopes to the sea (from east to west) in its entirety. So a dike ring area can flood from an already inundated upstream dike ring area when the separating dike fails (Figure 3.2). In The Netherlands, usually the separating dike is lower than the adjacent river dike.

Figure 3.2 Shortcut between dike ring areas: longitudinal section: B fails through A

Because dikes do not only fail from overflow but also from other mechanisms (paragraph 4.4.1), it is not unthinkable that flooding from the sea or a lake will cause the same effect, although the flooding could be against the inclination.

3 - Shortcut between rivers/river branches
Assume one dike ring area is surrounded by two rivers, for example one on the northern (river N) and one on the southern side (river S). Suppose that the dikes along the southern river are lower than those along the northern river (Figure 3.3). When the northern dike fails and the volume of inundation is larger than the storage volume of dike ring area A, water from the northern river flows through dike ring area A into the southern river which may cause very high water levels in that river. This will threaten other dike ring areas.

Figure 3.3 Shortcut between river: Cross-section: B fails through A

\(^{14}\) In this report both dikes are qualified as ‘separating dike’
It is clear that 1 (interaction through river) is a load reducing scenario because it might prevent failure of other dike ring areas. 2 (shortcut between dike ring areas) and 3 (shortcut between rivers) on the other hand are load increasing scenarios, as they induce new ways of failure, which are not accounted for in the isolated risk assessment. It is noted that in case of negative ways of interaction (2 and 3), the positive interaction (1) is still unabatedly operative: 2 and 3 always occur in combination with 1 (which will be explained in Table 3.2 on page 19).

Other ways of interaction
Other ways of interaction between dike ring areas are thinkable as well, e.g. interaction through tunnels (one inundated dike ring area can flood another one through a tunnel), indirect damage (inundation of one dike ring area can inflict damage to another non-inundated dike ring area due to economic 'links' in production processes or infrastructure). Interaction within one dike ring area between dike sections does exist as well: failure of one dike section might prevent failure of another dike section due to a fall of river water level (which resembles System configuration I). These other ways of interaction are not incorporated in this thesis.

Active - Passive
A distinction is made between an ‘active’ and a ‘passive’ approach of these interactions between dike ring areas. A passive approach means, that the (currently unknown) effects of the interaction between dike ring areas (in other words: the unknown differences between the currently applied isolated risk assessment and the integrated risk assessment) are implicitly accepted: nature/chance decides what will happen when a flood occurs. In an active approach, measures are taken to try to use the interaction between dike ring areas when floods are imminent in order to reduce the expected damage: the government/society tries to decide what will happen in case of a flood.

3.2.2 INTERACTION IN THE NETHERLANDS (HISTORICAL CONTEXT)

The distinguished ways of interaction in paragraph 3.2.1 are obtained by an analysis of the existing hydraulic (river) system in the Netherlands. History shows that these ways of interaction really did occur. Figure 3.4 on page 12 shows scenarios of interaction which occurred in historical floodings (a.o. 1784) in The Netherlands. Following disasters (in 1799, 1809 and 1820) showed the same development: a chain reaction from east to west. It has to be noted that several of these floodings were caused by so-called ice dams (which blocked the rivers and consequently rose their water levels) and that the dikes were less high and strong as they are today. In the PICASO-project it appears however that failure of the ‘Diefdijk’ (separating dike between dike ring area 16 and 43) is still a possibility despite the higher and stronger dikes and the absence of ice dams (the latter due to climate change and the discharge of cooling water in the rivers by industry). The shortcut between river Waal en river Maas through dike ring area 41 is also not unlikely because nowadays the dikes on the southern side along the river Maas are on average 2 m lower than the dikes on the northern side along the river Waal. The fact that during times of high water the discharge of the river Waal is about 3 times the discharge of the river Maas adds to the potential danger of this scenario. Another possible future scenario could be the flooding of dike ring area 14 (1/10,000) or 15 (1/2000) by river Lek or IJsselmeer via dike ring area 44 (1/1250).

15 Ven, van de, G.P. et al., Niets is bestendig, de geschiedenis van rivieroverstromingen in Nederland, Utrecht, 1995
16 Manen, van, S.E., PICASO deel I t/m V/I, 2001, shows a detailed description of a flooding risk calculation of dike ring area 43 (Betuwe, Tieler- en Culemborgwaarden)
Problem analysis

Figure 3.4 on page 12 only shows the negative ways of interaction (shortcuts between dike ring areas and river branches). However, the positive way of interaction (failure of one dike ring area prevents failure of another) was also very well known in the past. In the early days for example (paragraph 2.1), when organization of dike construction was still highly decentralized (each community protected its own area), people on one side of the river always tried to construct their dikes just a little higher than the dikes on the opposite side. It was thought that, when a high water came, the opposite side would flood first causing a fall of the river water level, which would prevent flooding on their side. Even stories of sabotage of dikes are known\textsuperscript{17}: a first and rather blunt application of water storage areas (as explained in paragraph 2.2.3).

Nowadays, possible effects of interaction between dike ring areas are (more or less unconsciously) taken into account as well: the expected Rijn discharge in 2050 (Table 2.1 on page 4) is a.o. based on future river works in Germany. The higher the dikes in Germany, the more water the Rijn in The Netherlands has to drain. It is expected that discharges higher than 18,000 m\textsuperscript{3}/s will not occur at Lobith due to extensive flooding in Germany.

\textsuperscript{17} Boer, E. de, ‘Morgen gebeurt het!’, Land + Water, ½ (2003), p.21.
3.3 **ISOLATED VERSUS INTEGRATED RISK ASSESSMENT**

3.3.1 **QUALITATIVE DESCRIPTION**

In current practice of determining the actual probabilities and risks of flooding (= risk assessment, see also Figure 2.3 on page 6) per dike ring area, the concerned dike ring area is (until now) regarded completely on its own and totally separated from other dike ring areas in the system.

In this ‘isolated’ assessment no interaction between dike ring areas is incorporated. In the determination of the hydraulic boundary conditions for a certain dike ring area (i.e. the probability of exceedance of a design high water level) for example, it is silently assumed that all other dike ring areas are infinitely strong and safe and will not fail. This does not seem very realistic as explained (Figure 3.1 on page 10). In addition, if the flood is not limited to one ‘isolated’ dike ring area, a ‘cascade-effect’ could occur (Figure 3.2 on page 10).

It is clear that this ‘isolated’ risk assessment highly simplifies the original system as it omits any interaction between dike ring areas, both positive and negative interaction. This is shown in Figure 3.5 on the basis of a fictitious hydraulic river system: the right side shows the simplifications that are made when dike ring area B is analysed according to the ‘isolated’ risk assessment.

![Figure 3.5 Simplifications of the system in the isolated risk assessment of dike ring area B (right). The black dots are possible points of failure](image)

Considering the preceding analysis it is assumed that the safety of a certain dike ring area depends on the safety of other dike ring areas, caused by the different ways of interaction between dike ring areas (sometimes called ‘system behaviour’ of dike ring areas18). It is expected that incorporating these interactions in the analyses will change the calculated probabilities and risks of flooding of dike ring areas. It is clear that these interactions can only be incorporated when the complete system is considered instead of just one isolated dike ring area. In order to express the difference with the isolated risk assessment of a single dike ring area, a risk assessment of the complete system is called an ‘integrated’ risk assessment. This is further explained in Figure 3.6.

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18 Delft Cluster project DC 02.01.01. Members: WL|Delft hydraulics, TNO Bouw, GeoDelft, TU-Delft and DWW.
The isolated risk assessment is ‘bottom-up’ (Figure 3.6): first each dike ring area is assessed separately, ignoring the existence of the other dike ring area in the system (Figure 3.5 on page 13) and the obtained probabilities and risks of flooding are combined to obtain the probability and risk of flooding of the system, although the latter is not explicitly executed in current practice.

The integrated risk assessment is ‘top-down’ (Figure 3.6): the system is assessed as a whole and the assessment results in probabilities and risks of flooding both for the system and for each dike ring area.

Recapitulating can be stated that a hydraulic system of rivers and dike ring areas can be assessed by means of an isolated and an integrated risk assessment. Both assessments can be applied on the total system and on a single dike ring area. As hydraulic interaction between dike ring areas is omitted in the isolated risk assessment and incorporated in the integrated risk assessment, the results of both assessments will differ.
3.3.2 QUANTITATIVE DESCRIPTION

In order to clarify the differences between the isolated and the integrated risk assessment, a simple example is treated based on Figure 3.7. In this configuration only interaction through the river plays a role (Figure 3.1 on page 10). First a qualitative examination is presented, followed by a quantitative analysis.

INTERMEZZO: LIMIT STATE FUNCTION

In order to indicate whether a structure (e.g. a dike) fails, a so-called limit state function is used, which in general form is expressed as $Z = R - S$, in which $R$ represents the strength of a certain structure and $S$ the load on that structure. Failure of the structure occurs if $S > R$, or if $Z < 0$. As both $R$ and $S$ are stochastic parameters, $Z$ is stochastic as well. Therefore the probability that $Z < 0$ can be calculated, which is the probability of failure of the structure (appendix C).

In this simplified example a dike only fails if the water height on the river exceeds the dike height (see also paragraph 4.4.1 for further explanation). Assume further:

- $Z_A = \text{limit state function of dike ring area } A$, idem for $B$;
- $R = \text{dike height};$
- $S = \text{water height on the river};$
- $D_A = \text{damage of } A \text{ in case of inundation, idem for } B;$
- $D_{AB} = \text{damage in case of inundation of both } A \text{ and } B.$

**Isolated risk assessment**

If the value of the limit state functions of $A$ and $B$ are displayed in a system of axis, the following figure is obtained (Figure 3.8). In the ‘top-right’ quadrant no failure will occur as both $Z_A$ and $Z_B > 0$; In the ‘top-left’ quadrant only $Z_A < 0$; in the ‘bottom-right’ quadrant only $Z_B < 0$; in the ‘bottom-left’ quadrant both $Z_A$ and $Z_B < 0$. If no interaction between dike ring areas is incorporated (which is the case in the isolated assessment), both dike ring areas will fail in this quadrant. It has to be noticed that the value of $P(A \cap B)$ is not known in the isolated assessment, as the total system is not considered. The probability and risk of flooding of the total system can therefore only be obtained by a combination of the probabilities and risk of the single dike ring areas. As the system of dike ring areas is considered as a serial system, this analysis yields for the total system:

$$P(\text{system}) = P(A) + P(B)$$  \hspace{1cm} (3.1)

$$\text{Risk}(\text{system}) = P(A) \cdot D_A + P(B) \cdot D_B$$  \hspace{1cm} (3.2)

19 Stochastic parameters are parameters with a statistical distribution such as a normal or exponential distribution.
Problem analysis

Integrated risk assessment

Consider again the ‘bottom-left’ quadrant in Figure 3.8. According to the isolated risk assessment both dike ring areas would fail in this quadrant, but if interaction through the river is incorporated, this will not always be true as failure of one dike ring area might prevent failure of another. The important issues are: which dike ring area will fail first and will therefore failure of the other dike ring area be prevented? In order to explain the complexity of the interaction, two simple assumptions are discussed first:

1. The weakest of the two dike ring areas will fail first and this failure will always prevent failure of the other (stronger) dike ring area.

   This assumption can be represented by a line in quadrant IV, which divides this quadrant into two areas: above the line holds: \( Z_A < Z_B \) and therefore A will fail (and therefore B will not fail); underneath the line it is the other way around.

2. The most upstream dike ring area (A) will fail first and this will always prevent failure of B.

   The difference with assumption 1 is the introduction of a time-aspect due to the propagation of the discharge wave through the river. In this assumption this time aspect is roughly applied as it is assumed that the most upstream dike ring area (A) is the one which will always fail first.

In reality the problem is more complex for several reasons:

- The dike ring area which fails first is not always necessarily the most upstream situated dike ring area: it might happen that the strength of a more downstream situated dike ring area (B) is exceeded before the upstream situated dike ring area (A) fails. This depends on the height and shape of the discharge wave, the strength parameters of the dike and the distance between the possible points of failure of the dike ring areas (more detail in paragraph 5.1.2).

- It is not known if failure of a dike ring area always prevents failure of another dike ring area. In order to determine if the second failure is prevented, it has to be examined what exactly happens to the river water level in case of failure.

These considerations could be displayed as in Figure 3.11. However, the subdivision of quadrant IV (\( Z_A < 0 \) and \( Z_B < 0 \)) is arbitrarily: whether a certain point in quadrant IV (\( Z_A, Z_B \)) belongs to a certain scenario (‘only A fails’, ‘only B fails’ or ‘A and B fail’) totally depends on the values of the strength and load parameters and on the behaviour of the river water in case of flooding. This is in contrast to assumption 1 and 2 in this paragraph, in which an exact subdivision could be made. It becomes clear that incorporating the interactions between dike ring areas accurately, hydraulic simulations are inevitable.
This analysis yields for the total system:

\[
P(\text{System}) = P(A) + P(B) - P(A \cap B) = P(A \cap B) + P(B \cap A) + P(A \cap B)
\]  
(3.3)

\[
\text{Risk(System)} = P(A \cap B) \cdot D_A + P(B \cap A) \cdot D_B + P(A \cap B) \cdot D_{AB}
\]  
(3.4)

The formulas for the isolated (formula 3.1 and 3.2) and integrated assessment (formula 3.3 and 3.4) differ for two reasons:

1. **Interaction in the probabilities of flooding**
   
   As the isolated assessment approaches each single dike ring area completely separately, the probability of failure of both A and B \( P(A \cap B) \) is by definition not known if the isolated assessment is applied. One is therefore forced to simply add \( P(A) \) and \( P(B) \) in order to obtain \( P(\text{system}) \); the isolated assessment leaves no other option (equation 3.1). In the integrated assessment \( P(A \cap B) \) is known and the probability of flooding of the total system can therefore be calculated more accurately (according to the theory of probabilities);

2. **Interaction in the inflicted damages**

   For the damage \( D_{AB} \) (the damage in case of failure of both A and B in the integrated assessment) holds: \( D_{AB} < D_A + D_B \). In case of a simultaneous failure water will flow into both dike ring areas, however, it is assumed that the river does not hold enough water to fill both A and B completely (paragraph 4.5 and appendix J). In addition, \( D_A, D_B \) and \( D_{AB} \) are not constant in reality but depend on the severity of the flood (Figure 3.12). This dependence will also be incorporated later in this thesis (paragraph 4.8).

![Figure 3.12 Difference between constant and variable damage](image)

**Example**

For the sake of clarity the distribution of the failure scenarios in the isolated and integrated assessment are once again presented (Figure 3.13).

![Figure 3.13 Distribution of the failure scenarios](image)

To clarify the implications of equation 3.1 to 3.4, a quantitative example is presented. Assume that by analysis of the system shown at the beginning of this paragraph (Figure 3.7 on page 15), the following probabilities and damages are obtained (Figure 3.14). The way in which such an analysis is performed, is subject of the coming chapters. The
values of the probabilities and damages in Figure 3.14 are fully fictitious. The way in which equation 3.1 - 3.4 have to be used in order to calculate the probabilities and risks of dike ring areas is shown in Table 3.1 on the basis of Figure 3.14.

![Figure 3.14](image)

**Table 3.1 Probabilities and risks of dike ring areas A and B and of total system**

<table>
<thead>
<tr>
<th>Dike ring area</th>
<th>Probability of flooding</th>
<th>Risk of flooding</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Isolated</td>
<td>Integrated</td>
</tr>
<tr>
<td>A</td>
<td>0.3</td>
<td>0.2+0.02+0.05= 0.27</td>
</tr>
<tr>
<td>B</td>
<td>0.4</td>
<td>0.3+0.03+0.05= 0.38</td>
</tr>
<tr>
<td>System</td>
<td>0.7</td>
<td>0.2+0.3+0.02+0.03+0.05= 0.6</td>
</tr>
</tbody>
</table>

Overall it can be concluded that hydraulic simulations are inevitable in an integrated risk assessment in order to obtain:
1. a realistic determination of the probabilities of flooding of dike ring areas and;
2. a realistic determination of the inflicted damage.

### 3.4 Problem Definition

The preceding analysis leads to the following definition of the problem:

The effects of the interaction between dike ring areas on the probabilities and risks of flooding of these dike ring areas are currently unclear.
Referring back to Figure 2.3 on page 6, the difference between the isolated and the integrated risk assessment mainly concerns (1): the probabilistic evaluation of strength and load. Due to the interaction between the dike ring areas this evaluation becomes more complex and will consequently have its effect on (2), (3) and therefore on (4).

### 3.5 OBJECTIVE AND DEMARCATION

#### 3.5.1 OBJECTIVE

The objective is defined as follows:

> To investigate (and acquire insight in) the quantitative effects of the interaction between dike ring areas on the probabilities and risks of flooding of these dike ring areas.

Comparing the results of the isolated risk assessment with the integrated risk assessment for different simple hydraulic systems will reveal these quantitative effects (Figure 3.6 on page 14). The hydraulic systems that will be assessed differ in configuration: the position of the dike ring areas in relation to each other and to the river varies. Each system configuration represents one or more ways of interaction (discussed in paragraph 3.2.1). The system configurations that will be analysed in this thesis are (schematically) displayed in Table 3.2. The different ways of interaction that play a role in each configuration are also shown. These system configuration will be discussed in more detail in chapter 5.

#### 3.5.2 DEMARCATION

As explained in paragraph 3.2.1 the number of ways in which interaction between dike ring areas can occur is limited to 3 (Table 3.2). The other ways of interaction are no subject of study in this thesis. Dike ring areas along the coast are not considered.

<table>
<thead>
<tr>
<th>Way of interaction</th>
<th>System configuration</th>
<th>1 Interaction through river</th>
<th>2 Shortcut between dike ring areas</th>
<th>3 Shortcut between rivers</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td></td>
<td>X</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
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Table 3.2 System configurations and ways of interaction ('X' = relevant, '-' = irrelevant)
The presented system configurations (Table 3.2) are completely fictitious. Although the system configurations are deducted from the Dutch situation (paragraph 3.2.2), they by no means intend to provide a detailed representation of the Dutch upper river system. Only some parameters and dimensions are roughly based on the situation in The Netherlands, in order to see to it that no unrealistic results are obtained.

In the objective of this report the effects of the interaction of dike ring areas are mainly related to the probabilities and risks of flooding of dike ring areas. However, the effects will also be of influence on the:

> probability of exceedance of design high water levels;
> economic optimization of dike ring areas and systems of dike ring areas;
> administrative organization.

These areas are no subjects of research in this thesis. However, for the sake of completeness they are discussed briefly in this paragraph.

**Probabilities of exceedance of design high water levels**

The protection standards as stated in the Flood Defence Act (appendix A) have to be applied on the primary water defence structures surrounding a dike ring area and are expressed as frequencies of exceedance of a design high water level. In the design and verifying activities regarding these structures, the formerly discussed isolated assessment is applied (Figure 3.6 on page 14). Including the discussed interaction in the design and verifying calculations will have its effect on the assumed levels of protection. However, the current shift in the philosophy of protection (from frequencies of exceedance of design high water levels to probabilities and risks of flooding of dike ring areas, paragraph 2.2.2) drives the probabilities of exceedance to the background.

**Economic optimization**

The results of the TAW research program (paragraph 2.2.2) should eventually lead to optimal protection levels for dike ring areas and consequently optimal dike heights by minimizing the total costs (appendix C). Despite the fact that the discussion about the future levels of protection of dike ring areas depends on other things as well (psychological, societal and political considerations), the economic optimum is still an important contribution to that discussion, especially when cost-benefit-calculations of measures in the system (such as dike improvement or emergency storage areas) have to be carried out.

Sub optimization per dike ring area (with or without incorporating interactions between dike ring areas) is not a correct approach: the system should be approached as a whole. A result of this optimization could also be an optimal level of protection of a separating structure, which is certainly not unambiguous at the moment.

**Administrative organization**

The interactions between dike ring areas will have its effect on the administrative organization regarding water management as well. The current administrative organization in the Netherlands is highly decentralized and responsibilities are assigned to local bodies, (water boards and provinces). Due to the interaction between dike ring areas local dike elevation will have its effect on the level of protection of structures in other areas. For the same reason assessment and maintenance of protection levels of the water defence structures will also become more complex.

Interaction between dike ring areas also influences evacuation plans under imminent flooding. In addition, possible future use of emergency storage areas will raise new issues in the decision process as well.
Problem analysis
4 MODELING

In this chapter the model is described which will be used to analyse the probabilities and risks of flooding of dike ring areas in simple hydraulic systems (the system configurations of paragraph 3.5.1). Paragraph 4.1 describes the assumptions on which the model is based, as well as the rough structure of the model. Paragraph 4.2 to paragraph 4.8 describe the different features of the model.

4.1 CONSIDERATIONS AND GENERAL DESIGN

In this paragraph the general (paragraph 4.1.1) and probabilistic (paragraph 4.1.2) considerations, which are the basis of the presented model are described. Paragraph 4.1.3 describes the general simplifications that are applied as well as a rough outline of the model.

4.1.1 GENERAL CONSIDERATIONS

What is needed?
It appears from the problem analysis (chapter 3, especially paragraph 3.3.2) that a correct analysis of the interaction between dike ring areas in an integrated risk assessment (Figure 3.6 on page 14) requires an accurate description of the water flow through the system; not only through the river, but also through the (inundated) dike ring areas. The ultimate aim would be the development of an integrated model of a complete river system, containing detailed modeling of rivers, dikes (and other hydraulic structures) and dike ring areas (geometry and economical values) and including all failure mechanisms of structures and uncertainties in model parameters. The model should reveal which scenarios of flooding can take place and should determine their probabilities and impacts. The effect of any measure in the considered area (e.g. dike strengthening, compartmentalization of dike ring areas) on the probability of failure and risk of various dike ring areas should easily be calculated.

What is available?
Detailed models are available which describe the separate hydraulic and geo-technical processes, which play a role in an integrated risk assessment: ‘Sobek’ describes the flow in the river system (one-dimensional), ‘PC-Ring’ describes the failure of dike rings, ‘Delft-FLS’ describes the inundation of dike ring areas, ‘HIS-SSM’[20] [Vrisou van Eck, 2000] based on ‘GIS’[21] determines the resulting damage and ‘Breach’ describes the development of a breach in a dike in case of failure. These programs (except PC-RING) are all deterministic, which means that no statistical distributions of parameters can be taken into account. A complete integrated model, which combines the various hydraulic and geo-technical processes, does not exist, although ‘Sobek 1D2D’ is able to combine the flow through the river and through the inundated dike ring area deterministically.

What is possible?
As will be explained in the next paragraph, the application of Monte Carlo Simulations is necessary. However, the incorporation of (integrated) hydraulic models (like Sobek) in Monte Carlo Simulations results in (relatively) accurate and flexible, yet very time-consuming calculations. These are not suitable when many different simulations must be

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performed. In order to enable a fast and usable Monte Carlo Simulation, concessions have to be made to the accuracy and flexibility: description of open channel flow, the inundation of dike ring areas, as well as failure mechanisms of hydraulic structures must be made less detailed and complex. Only then an integration of these processes in a Monte Carlo Simulation is possible.

### 4.1.2 Probabilistic Considerations: Monte Carlo

#### What is needed?

In case only the economical damage is considered (paragraph 2.2.2), risk is generally defined as:

\[
R = P_f \cdot D, \tag{4.1}
\]

where (in case of a dike ring area):

- \( R \) = Risk [€/Year];
- \( P_f \) = Probability of failure [1/Year] (calculated by an evaluation of all failure mechanisms (paragraph 4.4.1) and stochastic parameters);
- \( D \) = Damage [€].

Examples of stochastic parameters are: height and shape of the discharge wave, dike dimensions, soil properties, wind speed, roughness of riverbed and dike ring area etc. In case of a (separate) dike ring area however, the damage ‘D’ highly depends on the stochastic parameters, the number of dike failures and their locations and is therefore not a constant value (Figure 3.12 on page 17). When a system of dike ring areas is being regarded, the physical interaction between the dike ring areas (paragraph 3.2.1) even more increases the dependence of both probability of failure and risk of the stochastic parameters. Equation (4.1) is no longer suitable for a flooding risk calculation. An expression, which satisfies the dependence of both probability of failure and risk of the stochastic parameters, would be:

\[
R = \int f(x) \cdot D(x) \cdot dx, \tag{4.2}
\]

where:

- \( x \) = vector containing all the stochastic parameters (theoretically the stochastic parameters may also be time-dependent, but this is not considered here);
- \( f(x) \) = probability density function of \( x \).

In order to solve equation (4.2), deterministic hydraulic simulations for all the combinations of \( x \) (resulting in a damage per dike ring area) have to be performed and combined. In these deterministic simulations all the hydraulic interactions can be accounted for.

#### What is possible?

Equation (4.2) is not applicable on a realistic hydraulic system; not only because an integrated model combining all the geo-technical and hydraulic processes does not exist, but also because the almost infinite amount of simulations that should be performed as well as the calculation time for each simulation. So simplifications have to be made in order to solve the equation.

The way the equation is going to be solved is by means of a Monte Carlo simulation. In a Monte Carlo simulation a sampling of the distribution functions of the stochastic
parameters (strength and load) is performed which results in a number of deterministic scenarios or ‘runs’ with different values of the strength and load parameters (e.g. maximum river discharge and dike height). For each run the response of the hydraulic system is determined: where does failure occur and what is the resulting damage? This is the application of the hydraulic simulations of which the necessity has been demonstrated in paragraph 3.3.2. Now the probability of failure and risk of a dike ring area (e.g. A) is calculated as follows:

\[
P_f(A) = \frac{p_A}{N} \quad \text{and} \quad \text{Risk}(A) = \frac{1}{N} \sum_{i=1}^{N} D_{A,i},
\]

where:
N = Number of Monte Carlo runs;
p_A = Number of Monte Carlo runs in which failure of dike ring area A occurs;
D_{A,i} = Damage of dike ring area A in run i.

In the same way the probability of failure and risk of the system can be calculated: p_A then becomes p_{system} and D_{A,i} becomes D_{system,i}.

Considering equation (4.3) it appears that the probability of failure and risk are only calculated accurately enough when the number of runs is sufficiently large (more information in appendix E). Again it is stated that such a Monte Carlo simulation is for the time being only possible in a highly simplified system. These simplifications are discussed in the following paragraphs (4.2 to 4.8).

4.1.3 SIMPLIFICATIONS AND OUTLINE

The simplifications that are applied can be subdivided in the following categories:
> Hydraulic simplifications;
> Geo-technical simplifications;
> Probabilistic simplifications.

Hydraulic simplifications
A hydraulic model like Sobek performs a detailed calculation of the flow-process in time by integrating the differential equations of De Saint Venant, taking both the friction and the inertia into account (appendix F). The simulations in time of the flow through the river, the flow through a developing breach after failure and the flow through the dike ring area as well as the level of detail make the model quite cumbersome. The Saint Venant equations can be reduced in complexity and can be solved analytically, when plausible simplifications are made. The discharge through the developing breach and therefore the volume of inundation can be estimated without the use of simulations in time (at least in the Monte Carlo simulation itself) as well. The lowering of the discharge wave and the occurrence of translation waves in the river are also important in the flow process (as will be explained). Simplified approximations for these phenomena are developed as well.

Geo-technical simplifications
As will be explained dike rings can fail due to several failure mechanisms. When failure occurs, a breach will develop in the dike. In reality these breaches can occur at any location in the dike. In this model possible breach locations will be limited. The number of failure mechanisms that will be accounted for will also be reduced, as well as the level of detail of the geo-technical description of these mechanisms. The dike ring area will only consists of dikes; neither dunes nor special structures are considered.
Probabilistic simplifications
In theory all the parameters in the model should be stochastic, because that would be
the best representation of reality: every variable is more or less uncertain. Again for
reasons of simplicity the number of stochastic parameters in this model will be limited
to the most important ones. The most important load and strength parameters will be
stochastic. The damage of a dike ring area will be determined in a rough way.

Outline of the model
Although some accuracy and flexibility will be lost in relation to e.g. Sobek, the model
will be able to calculate simple hydraulic systems. A computer program written for this
purpose in Turbo Pascal 7.0 (appendix M for the code) performs sampling of the
distribution functions by using a random generator (appendix M for the formulas). The
values of the parameters of the runs that will lead to failure are written in a file, which is
being imported and processed in Excel. This outline is schematically displayed in Figure
4.1.

4.2 OPEN CHANNEL FLOW
In this paragraph the flow of the water in the river is described. First the river used in
the model is described (paragraph 4.2.1) and second the different flow equations are
presented (paragraph 4.2.2).

4.2.1 RIVER
Figure 4.2 shows a typical cross-section of a Dutch river. The dike ring area is separated
from the river by the winter dikes, which retain the high waters generally occurring
during winter.

For reasons of simplicity the cross-section of the river in the model will be as in Figure
4.3. The river is perfectly straight: no meanders exist. The land level of the dike ring
areas is equal to the water depth corresponding with \( Q_{\text{average}} \). Both the bottom of the
river as well as the bottom of the adjacent dike ring areas slope to the west. The values
of the variables are shown in Table 4.1. The geometry of the river as well as the
discharges (paragraph 4.3) are purely fictitious although roughly based on the river Waal
in The Netherlands.
4.2.2 Flow

Discharge - water depth relation

The relation between the river discharge \( Q \) and the waterdepth \( d \) can be described by the following equations (background in appendix F):

\[
Q = C \cdot A \cdot \sqrt{R \cdot i}, \tag{4.4}
\]

where:

\[
C = \frac{g}{c_f} = 18 \cdot \log \left[ \frac{12 \cdot R}{k} \right]
\]

\[
A = d \cdot w
\]

\[
R = \frac{d \cdot w}{2 \cdot d + w}. \tag{4.5}
\]

Table 4.2 describes the different variables. Because of the complexity of the expression the relation is approached by a power-function with a Least Square Method (Figure 4.4, the blue dots are the results of formulas 4.4 and 4.5).

Celerity of propagation

It can be shown (background in appendix F) that, when the roughness of the river varies with the water depth according to Strickler or Manning \((c_f \sim d^{-1/3})\), the celerity of propagation becomes:

\[
c_{HW} = \frac{5}{3} \cdot U, \text{ where } U = \frac{Q}{A}. \tag{4.6}
\]
4.3 DISCHARGE WAVE

This paragraph describes the discharge wave. First the height (or maximum discharge, paragraph 4.3.1) and second the shape (paragraph 4.3.2) of the discharge wave is determined. Because the geometry of the river in this model is roughly based on the river Waal in The Netherlands, the discharges are also roughly based on the situation in The Netherlands in order to create a realistic system.

4.3.1 HEIGHT (MAXIMUM DISCHARGE)

The probability distribution function of the maximum discharges ($Q_{\text{max}}$) is determined in [HR2001] for the river Rijn at Lobith (on the Dutch-German border). It is assumed that $2/3$ of that discharge flows through the river Waal. In this model the design discharge $Q_{\text{max,design}}$ (occurs once in 1250 years) is multiplied by a ‘discharge factor’ $\delta_{\text{discharge}}$, which is exponentially distributed in such a way, that it approximates the distribution of [HR2001]. The formula of the discharge factor is:

$$\delta_{\text{discharge}} = \frac{Q_{\text{max}}}{Q_{\text{max,design}}}.$$  

The formula of the probability distribution function of $\delta_{\text{discharge}}$ is:

$$P(x) = e^{-x/0.08228} \quad P(x) \text{ is probability of exceedance of } x. \quad (4.8)$$

System configuration III (paragraph 5.3) consists of two rivers, river N(orth) and S(outh). The geometry of both rivers is the same, but their discharges differ ($Q_{\text{max,design}}$ of river N is 10666 m$^3$/s and of river S 9020 m$^3$/s). Figure 4.5 shows the probability distribution function for the maximum discharges.

![Figure 4.5 Probability distribution function of maximum discharge in the fictitious river](image)

Figure 4.5 Probability distribution function of maximum discharge in the fictitious river

---

4.3.2 Shape

Next to the height of the discharge wave, the shape (development in time) is important as well, because it determines the effect of the more time-dependent failure mechanisms (paragraph 4.4.1) as well as the volume of inundation (paragraph 4.5). However, to predict the shape of a certain discharge wave with a maximum discharge that has not occurred in reality yet, is very difficult. In [HR 2001] the expected shape of the discharge wave corresponding with the ‘design discharge’ in Lobith has been determined. Because the fictitious river in this model is roughly based on the river Waal, the discharge wave of this model is based on the expected discharge wave of the river Waal (which is obtained by assuming that 2/3 of the discharge at Lobith flows through the river Waal). The blue line in Figure 4.6 shows the design discharge wave for the river Waal. The red line shows the simplified approximation used in this model. The total volume contained by these waves is roughly equal. It is assumed that a discharge wave occurs only once a year. During the rest of the year the discharge is 1500 m$^3$/s (Figure 4.7). For every $Q_{max}$ the shape is multiplied by the already explained discharge factor. The duration of the wave does not change. (The formulas which describe the shape of the wave and are used in the calculations are presented in appendix G.)

![Design discharge wave of river Waal and simplified approximation in model (T1=400 h, T2=600 h)](image)

![Different occurring discharge waves in time](image)
4.4 FAILURE OF DIKES

In this paragraph the various ways in which dikes can fail are described. In paragraph 4.4.1 the different failure mechanisms, which are implemented in this model are described and paragraph 4.4.2 deals with the subsequent breach growth.

4.4.1 FAILURE MECHANISMS OF RIVER DIKES

As already explained the water defence structure enclosing a dike ring area may consist of dikes, dunes, special structures and high grounds. In this model only dikes enclose a dike ring area. Experience has demonstrated [Picaso 2000] that the most important failure mechanisms of dikes is overflow/overtopping. In this model, piping is implemented as well as will be explained (Figure 4.8). Several other failure mechanisms exist as well, e.g.: settlement, instability of outer slope, instability of inner slope, collision by a vessel etc. These mechanisms are not accounted for in this model.

\[
H_{\text{river}} > H_{\text{cr,overflow}} \quad \text{‘cr’ = critical}
\]

\(H_{\text{cr,overflow}}\) is distributed normally, with a standard deviation of 0.3 m. This standard deviation does not only represent the fluctuation in dike height, but also the fluctuation in wave attack on different locations and uncertainties in the Q-d relation which always exists in reality. This means that uncertainties in load-parameters are represented by standard deviations in strength-parameters. Despite the fact that normally this is not a proper way of modeling, it is applied for the sake of simplicity.

2. Piping

In case of piping the water underneath the (more or less) impermeable clay layer will build up pressure due to the increasing head difference. When the clay layer bursts at a weak spot (e.g. at a ditch) the water will start to transport sand from under the dike to the well. A pipe under the dike will start to develop which will undermine the dike after a certain period of time and a gap or breach is formed. So in addition to the river water...
level (or head difference), piping also strongly depends on time. Therefore in this model failure occurs when:

\[ H_{river} > H_{cr,piping} \text{ AND } |t - t_0| > T_{cr,piping}, \]  

(4.10)

where \( t_0 \) is the moment when for the first time \( H_{river} > H_{cr,piping} \cdot H_{cr,piping} \) and \( T_{cr,piping} \) are both distributed normally. \( H_{cr,piping} \) has a standard deviation of 0.5 m (mean is varied in the different calculations). The value of \( T_{cr,piping} \) is very hard to estimate as little is known about the time development of failure mechanisms in general. It is known however that once a pipe is formed, collapse occurs very fast. One case of dike failure due to piping in Groningen\(^{23}\) shows that a pipe of considerable length can develop in half a day. The parameters of the distribution of \( T_{cr,piping} \) are therefore estimated as follows: mean 6 h, standard deviation 3 h.

In Figure 4.9 the above discussed failure mechanisms are presented in a fault tree. In this model failure can only occur on predefined locations. Only one failure mechanism per location can occur: the mechanisms exclude each other on one location. Formulas of the time of failure of overtopping and piping are to be seen in appendix H. According to the design rules\(^{24}\), the probability of failure due to piping must be <10% of the probability of failure due to overtopping. This rule will be applied in the calculations. Actually both mechanisms should be calculated with the use of far more accurate models including wind celerities, fetch, overtopping discharge, soil parameters and erosion and piping formulas. For the sake of simplicity it is refrained from such detailed calculations in this model.

\[ \text{Dike failure} \]

\[ \text{Overflow} \]

\[ \text{Piping} \]

\[ H_{river} > H_{cr,overflow} \text{ or } H_{river} > H_{cr,piping} \]

\[ |t - t_0| > T_{cr,piping} \]

Figure 4.9 Fault tree of dike failure

4.4.2 BREACH GROWTH

As explained above, both failure mechanisms create an initial breach when failure occurs. Due to the induced head difference water starts flowing through the breach and because of erosion the breach will widen. The duration of the development of the (vertical) initial breach as well as the duration of the subsequent (horizontal) widening depends on many factors, such as the material of the upper dike layer (e.g. grass), the core material (sand, clay) and on the (initial) head difference. Ideally, combining these variables in an erosion formula should simulate the breach growth. However, due to many uncertainties breach growth variables in flood modeling are usually input variables instead of results of calculations. In experiments\(^{25}\) it is shown that the horizontal breach growth can be described according to the following formula:


4 Modeling

\[ B_{\text{breach}}(t) = B_{\text{max}} \cdot \sqrt{\frac{t}{T_{\text{end}}}}, \]  
(4.11)

where \( B_{\text{max}} \) is the final breach width and \( T_{\text{end}} \) the time interval in which \( B_{\text{max}} \) is reached.

Estimations of values of \( B_{\text{max}} \) in river dikes of clay vary considerably: from 40 - 200 meter to even 2500 meter. The same holds for \( T_{\text{end}} \), which varies from 24 to 200 h. Considering historic breaches in clay dikes, which were usually about 100 a 300 meter, in this report \( B_{\text{max}} \) has a value of 150 meter and \( T_{\text{end}} \) of 60 hour. It is also assumed that when failure occurs, the horizontal breach immediately starts to develop over the total height of the dike.

4.5 VOLUME OF INUNDATION

Calculation of the volume of water, which flows into a dike ring area, is important as it determines:

1. the damage that is inflicted by the inundating water to that dike ring area (see also paragraph 4.8);
2. whether the flood is confined to one dike ring area or if the water will flow into other dike ring areas or river branches.

In paragraph 4.5.1 the volume of inundation in case of an uncontrolled dike breach is discussed followed by the description of the volume of inundation in case of the application of an emergency storage area in paragraph 4.5.2. In paragraph 4.5.3 the storage volume of a dike ring area is calculated.

4.5.1 UNCONTROLLED DIKE BREACH

In case of an uncontrolled dike breach it is assumed (see also PICASO [2001]) that the discharge through the breach can be modeled by the following formula:

\[ Q_{\text{breach}}(t) = \mu \cdot \Delta Z(t) \cdot B_{\text{breach}}(t) \cdot \sqrt{2 \cdot g \cdot \Delta Z(t)}, \]  
(4.12)

where \( \Delta Z(t) \) is the momentary head difference, \( \mu \) is the contraction coefficient (value = 1) and \( B_{\text{breach}}(t) \) varies according to the formula mentioned in paragraph 4.4.2. The discharge can be schematized as follows (Figure 4.10):

\[ Q_{\text{in}} = Q_{\text{out}} + Q_{\text{breach}} \]

![Figure 4.10 River and discharge through the breach](image)

---

By applying this formula it is assumed that the water level behind the dike does not influence the discharge through the breach; $\Delta Z(t)$ is therefore the difference between the momentary river water level and the land level in the dike ring area. This assumption is only accurate when the water behind the dike can easily flow off, e.g. in case of a breach at the top of an inclining dike ring area. When a breach occurs at the bottom of an inclining dike ring area or when the dike ring area is very small, the water level behind the breach will rise quickly (Figure 4.11). In those cases this formula can no longer be applied.

In the Monte Carlo Simulations it is important that for different combinations of discharge waves and dike heights, the volume of inundation is calculated correctly. In Figure 4.12 the results of these volume calculations are presented. It appears that the later the failure occurs, the smaller the volume of inundation. The decreasing lines represent failure during rising water level, the increasing lines failure during falling water level. The blue dots in this figure display the solutions following from equations (4.11) and (4.12); the red lines represent a simplified approach which is extensively described in appendix I. The main point of this simplified approach is in short displayed in Figure 4.13: the (complex) course of the discharge through the breach (blue lines, results of formula 4.11 and 4.12) is approached by two straight lines (red), in such a way, that the total volume of inundation (in Figure 4.13 represented by the area enclosed by the blue and black line) is equal. This simplified approach has been developed in order to avoid extensive simulations in time in a Monte Carlo simulation, which would consume a lot of calculation time. The volume of inundation is now calculated by a set of straightforward formulas (appendix I).
This calculation of the volume of inundation is based on the initial shape of the discharge wave. However, after the first failure the shape changes drastically. The volume of inundation in case of a second failure is therefore calculated differently (appendix J).

It has to be noted that in the calculation of the ‘exact’ course of the discharge through the breach, hysteresis (appendix F) has not been accounted for. This could be of influence on the absolute volume of water, which flows into a dike ring area. The trends as discussed in this paragraph will stay the same, as the absolute deviation of the volume of inundation as a result of the hysteresis will be approximately the same for each situation.

4.5.2 EMERGENCY STORAGE AREA

Considering the geographical situation in The Netherlands (densely populated), future emergency storage areas will be relatively small and therefore only able to store the peak of the discharge wave. This makes heavy demands on the design and operation of inlet works of emergency storage areas. In this model it is assumed that the inlet work is designed as a weir with a threshold (height of threshold corresponds with $Q_{cr,esa}$). If $Q_{river} > Q_{cr,esa}$, the discharge surplus ($Q_{river} - Q_{cr,esa}$) will flow into the dike ring area. This effect is called: ‘peak shaving’ (Figure 4.14) and the volume ‘$V$’ is now calculated as follows:

$$V = \frac{1}{2} \left(Q_{max} - Q_{cr,esa}\right) \cdot \alpha \cdot (T_1 + T_2)$$

where $Q_{max}$ is the maximum discharge, $Q_{cr,esa}$ is the critical discharge for emergency storage area, $T_1$ and $T_2$ are the times, and $\alpha$ is the coefficient:

$$\alpha = \frac{Q_{max} - Q_{cr,esa}}{Q_{max} - Q_{av}}.$$

(4.13)
4.5.3 **STORAGE VOLUME DIKE RING AREA**

As already mentioned (paragraph 4.2.1) the bottom of a dike ring area adjacent to the river has got the same slope as the bottom of the river (there is no inclination in the direction perpendicular to the river). This has got important consequences for the storage volume of the dike ring area. A cross-section of an inundated dike ring area is shown in Figure 4.15.

![Figure 4.15 Cross-section inundated dike ring area](image)

The storage volume is determined by the height of the separating dike at the west side of the dike ring area: in The Netherlands separating dikes are usually lower than the adjacent river dikes. The storage volume is now calculated as follows:

\[
 x < L: \quad V = \frac{1}{2} \cdot \frac{H_{cr,overflow}^2}{i_b} \cdot B_{area} \quad \text{4.14}
\]

\[
 x > L: \quad V = \frac{1}{2} \cdot L^2 \cdot i_b \cdot B_{area} + \left( L \cdot i_b - H_{cr,overflow} \right) \cdot L \cdot B_{area} \quad \text{4.15}
\]

where \( B_{area} \) is the width of the dike ring area in the direction perpendicular to the river.

In this approach the so-called 'static storage' is applied, which means that the surface of the inundated water in the dike ring area is always horizontal and no flow through the dike ring area does occur. Another possibility would be 'dynamic storage' in which the water is able to keep flowing through the dike ring area because on the down side of the area the water is guided back to the river through a local lowering in the dike. This is only possible if the river water level at the location of the lowering is lower than the water level in the inundated dike ring area. An advantage of the application of dynamic storage is, that the storage capacity is larger than in the case of static storage. However, for reasons of simplicity, dynamic storage is not applied in this model.

4.6 **LOWERING OF THE DISCHARGE WAVE DURING PROPAGATION**

The shape of the initial discharge wave of paragraph 4.3 will change during propagation. Due to hysteresis (appendix F) the wave will become lower and wider. In this paragraph this distortion is investigated in the hydraulic software package ‘Sobek’ and a method is developed which describes the lowering of the discharge wave in a straight forward way, avoiding simulations in time, which would prolong the duration of the calculation considerably. The geometry of the river in the program is according to paragraph 4.2.1 (length of the river in the program is 320 km) and the distortion is investigated for various discharge waves and moments of failure. Every 40 km the shape of the discharge wave is measured.
In general two different situations can be distinguished:
1. Distortion in case of an uncontrolled dike breach (paragraph 4.6.1);
2. Distortion in case of the application of an emergency storage area (paragraph 4.6.2).

4.6.1 UNCONTROLLED DIKE BREACH

When a discharge wave propagates through a river and no measures are taken by government/society (like the application of emergency storage areas and/or by-passes), nature/chance decides if, where and when dike failure will occur (paragraph 3.1, 'active-passive'). Four different situations can then be distinguished:
1. No failure occurs;
2. Failure occurs in the top;
3. Failure occurs during rising water level;
4. Failure occurs during falling water level.

In Figure 4.16 the distortion of a discharge wave in case 1 (no failure) and 2 (failure in the top\(^{29}\)) is shown. The line on the left in each figure is the initial discharge wave (as discussed in paragraph 4.3) and the lines in between show the shape of the discharge wave at intervals of 80 km. It can be seen that the top of the wave lowers during propagation and due to the conservation of volume, the wave becomes wider as well. In Figure 4.17 the lowering of the top of the discharge wave (encircle in Figure 4.16) is displayed as a function of the distance. It is to be seen that the remaining top after failure lowers more than in case of no failure.

---

\(^{29}\) Due to stability problems in Sobek, the course of the discharge wave at \(x = 0\) km at the time of failure is not totally vertical, which should be the case considering the approximation of the course of the discharge of Figure 4.12 (red line). The steep sloping line will somewhat reduce the lowering speed of the top of the wave. It is added that the original course of the wave (Figure 4.12, blue line) is not completely vertical either.
Figure 4.17 Lowering of discharge wave in case of no failure and of failure in the top for various values of $Q_{\text{max}}$

The lowering of the top as a function of the distance along the river can be approximated (the approach of the fitting procedure is the same as in appendix I) by a tangent hyperbolic (values of parameters in Table 4.3, ‘$x$’ is the distance along the river (km) and failure occurs at $x = 0$ km):

$$\Delta Q_{\text{max}} = (c \cdot Q_{\text{max}} + d) \cdot \tanh\left((e \cdot Q_{\text{max}} + f) \cdot x\right),$$

4.16

Table 4.3 Parameters of formula 1.23, no failure

<table>
<thead>
<tr>
<th>Parameter</th>
<th>No failure</th>
<th>Failure in top</th>
</tr>
</thead>
<tbody>
<tr>
<td>$c$</td>
<td>-3.90*E-8</td>
<td>-6.47*E-8</td>
</tr>
<tr>
<td>$d$</td>
<td>+7.30*E-3</td>
<td>+4.52*E-3</td>
</tr>
<tr>
<td>$e$</td>
<td>+2.99*E-2</td>
<td>+6.06*E-2</td>
</tr>
<tr>
<td>$f$</td>
<td>+1.36*E+2</td>
<td>+1.85*E+2</td>
</tr>
</tbody>
</table>

The lowering of the discharge wave is in the order of decimeters, which is in the same order as the variation in dike height (paragraph 4.4.1).

It appears that the course of lowering of the discharge wave in case of no failure (1) and of failure in the top (2) can be described by a rather straightforward tangent hyperbolic function as a function of ‘$x$’ and ‘$Q_{\text{max}}$’. However, when failure occurs during rising (3) or falling (4) water level the situation becomes far more complicated: the lowering course does not only depend on ‘$x$’ and ‘$Q_{\text{max}}$’, but also on ‘$Q_{\text{failure}}$’. The following solution is found (details in appendix K):

> When failure occurs during rising water level, the course of lowering of the remaining top is approached by the course of lowering in case of failure at the top of the same wave;

> When failure occurs during falling water level, the course of lowering of the top is approached by the course of lowering in case of no failure of the same wave.

In order to check whether the course of lowering of the simplified (triangular) discharge wave is realistic, a comparison is made with the lowering course of the design discharge...
wave of paragraph 4.3.2. It appears that the two courses of lowering do not deviate much (appendix K).

4.6.2 EMERGENCY STORAGE AREA

In this paragraph the distortion of a discharge wave after the use of an emergency storage area is investigated (see also paragraph 4.5.2). In this simple example it is assumed that an emergency storage area is brought into action when the discharge exceeds 10,000 m$^3$/s. It is further assumed that the discharge wave is topped according to the 'peak shaving' as explained in paragraph 4.5.2. In Figure 4.18 the distortion of a discharge wave with a top of (initially) 11,000 m$^3$/s after the use of an emergency storage area is shown.

![Figure 4.18 Distortion of discharge wave after 'peak shaving' by emergency storage area](image)

It can be seen from Figure 4.18 that even though the peak of the wave becomes a little smaller, actual lowering of the top of the wave does not occur (at least not in the range 0 to 320 km which is covered by this figure). Higher waves will have wider tops after 'peak shaving' (when the critical discharge is constant) and will lower even more slowly. In this model it is assumed, that the shape and the height of a discharge wave after 'peak shaving' will not change during propagation.

4.7 TRANSLATION WAVES

Next to the friction dominated discharge wave, which varies slowly in time and space, another wave can propagate through the river as well: a so-called translation wave. Translation waves are caused by sudden changes in discharge/water depth, which can result from e.g.: the sudden closure of a weir in a river, the sudden opening of a door in a lock and the sudden failure of a dike. Translation waves induced by dike failure are negative waves which means that it is a local lowering of the water level compared to the surrounding river water level. As these waves propagate through the river system, they might prevent failure of other areas in the system. In contrast to the discharge wave, friction does not play an important role in the propagation of a translation wave: inertia is far more important, which is shown in the expression of the celerity of propagation of a translation wave:

$$c_t = \sqrt{g \cdot d}, \quad 4.17$$

where ‘d’ is the waterdepth and ‘g’ is the acceleration of gravity. It is useful to compare the $c_t$ (velocity of propagation of translation wave) with $U$ (flow velocity) and $c_{hw}$ (velocity of propagation of high water wave). The flow velocity varies from about 0,9 m/s ($Q = 1500$ m$^3$/s) to 2 m/s ($Q = 11000$ m$^3$/s). The velocity of propagation of the high water wave then varies from about 1,4 m/s to 3 m/s. Assume that after a certain dike failure the water level drops to about 5,5 m, then the velocity of propagation of the translation wave becomes 7,3 m/s. In principle (standard solution of the wave equation) a translation wave propagates in two directions: upstream and downstream. Because $c_t$
4 Modeling

is larger than both $U$ and $c_{ws}$, the translation wave can indeed travel in the upstream direction. The upstream and downstream velocities are expressed as follows:

\[
c_{tr,\text{upstream}} = U - \sqrt{g \cdot d} \quad (4.18)
\]
\[
c_{tr,\text{downstream}} = U + \sqrt{g \cdot d} \quad (4.19)
\]

In this model the depth $d$ corresponds with $Q^*$ in appendix I.

4.8 Consequences of flooding

As explained in paragraph 2.2.2 the damage in a dike ring area inflicted by inundation from a major river can be manifold:

- Economical damage;
  - Direct: damage to capital goods etc;
  - Indirect: loss of production and income;
- Casualties;
- Damage to the environment;
- Psychological damage.

In this model only economical damage is taken into account. In the Standard Method\(^{30}\) the damage depends on three variables: the maximum water depth, the maximum flow velocity and the velocity of the rising water. In a detailed calculation of the damage the dike ring area is divided into a large number of grid cells (100 x 100 m) and for each grid cell the three variables are determined (by a detailed simulation of the inundation) and a “damage factor” (value between 0 and 1) is calculated. The $D_{\text{max}}$ of the considered grid cell (the maximum damage that can be inflicted, based on the replacement value of the capital goods) is multiplied by the damage factor and the damage of the considered grid cell is determined. In the calculation of the damage factor a distinction is made between different kinds of land use (agriculture, buildings etc.) The sum of the damage for each grid cell over all the grid cells gives the damage of the dike ring area.

The subdivision of a dike ring area in grid cells is too detailed for this model considering the roughness of the modeling of the (flow through the) dike ring area. The following relation is adopted:

\[
D = \frac{V_{in}}{V_{\text{initial, cap}}} \cdot D_{\text{max}}, \quad 4.20
\]

where $D =$ damage, $V_{in} =$ volume of inundation, $V_{\text{initial, cap}} =$ storage volume of dike ring area and $D_{\text{max}} =$ maximum damage.

Increase of the storage volume of a dike ring area (e.g. by strengthening of a separating dike) has no influence on ‘$D$’, because in the calculation of ‘$D$’, the initial storage volume is used. Therefore any later increase of the storage volume has only effect on the overflow of the concerned dike ring area (the larger the storage volume, the less overflow will occur). After dike strengthening the maximum damage does no longer necessarily coincide with overflow.

\(^{30}\) Vrisou van Eck, N. en M. Kok, *Standaardmethode Schade en Slachtoffers als gevolg van overstromingen*, Delft 2000
In this chapter each of the three ways of interaction (paragraph 3.2.1) between dike ring areas are converted into small hydraulic systems consisting of rivers and dike ring areas. Such a small system is denoted as 'system configuration'. In this chapter the (three) system configurations are described. It starts with a general description followed by an analysis of the respective system configuration. In the figures of the system configurations in this chapter, dike ring area A is always red and B blue. In the presentation of the results in the next chapter these colors are used as well and refer to the respective dike ring areas.

5.1 System Configuration I - Interaction through river

5.1.1 Description

System configuration I (Figure 5.1) represents a system consisting of one river and two dike ring areas. The river flows from east to west and has the discharge statistics of river N in paragraph 4.3.1. Dike ring area A is situated on the northern side of the river and B on the southern side. The whole system slopes to the west (1m/10km). The dike ring areas do not slope in the direction perpendicular to the river. The land level in the dike ring areas is situated 3 m above the bottom of the river. Each dike ring area has got one possible point of failure. The distance between these points is denoted with ‘x’ (the distance is varied in the calculations). Both dike ring areas are equally large and have a length of 65 km and a width of 41.9 km. The separating dikes on the west side of both dike ring areas have a height of 6.0 m compared to the land level in the dike ring areas (9.0 m compared to the bottom of the river). This gives a storage volume of $7.57 \times 10^9$ m$^3$. The background of the choice of these dimensions will be discussed in paragraph 5.2.1. The maximum potential damage$^{31}$ for A is € $27 \times 10^9$ and for B € $54 \times 10^9$. When A or B overflows (i.e. the volume of inundation in A or B is larger than the storage volume of A or B, the water flows out of the system without inflicting any more damage. It has to be noticed that in a realistic river system (such as Figure 3.4 on page 12) this assumption is not very realistic.

![Figure 5.1 System configuration I](image_url)

5.1.2 Analysis

As explained in paragraph 3.3.2 (page 15), the interaction between dike ring areas starts to play a role when initially in the isolated risk assessment both dike ring areas would fail (both $Z_A<0$ and $Z_B<0$). It was concluded in paragraph 3.3.2 that the interaction can only be incorporated when a hydraulic simulation is performed. The hydraulic model developed in chapter 4 and incorporated in a Monte Carlo simulation enables such a

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$^{31}$ These values are more or less based on dike ring area 43: Doef, van der, M.R., *Pilot Case Overstromingsrisico’s, Deel V: Berekenen van schade en slachtoffers*, Delft 2001
detailed analysis. In a Monte Carlo Simulation as described in paragraph 4.1.2 the hydraulic response of the system in each single run can be determined which dike ring areas will fail and what will be the eventual damage?

In order to investigate the effects of the interaction between the dike ring areas it has to be determined which dike ring area fails first, due to which failure mechanism and how and if failure of the other dike ring area is prevented. Theoretically 18 scenarios for which $Z_A<0$ and $Z_B<0$ can be derived. These scenarios are displayed in Table 5.1 as well as the way in which failure of the other dike ring area could be prevented by the changing hydraulic conditions.

Table 5.1 Possible scenarios for which both $Z_A$ and $Z_B$<0

<table>
<thead>
<tr>
<th>#</th>
<th>First failure</th>
<th>Second failure</th>
<th>Second failure is prevented if: …</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Dike ring area</td>
<td>Failure mechanism*</td>
<td>Dike ring area</td>
</tr>
<tr>
<td>1</td>
<td>Dike ring area</td>
<td>Overtopping</td>
<td>the remaining top after failure A lowers below $Q_{\text{failure,overflow,B}}$</td>
</tr>
<tr>
<td>2</td>
<td>Dike ring area</td>
<td>Piping rising</td>
<td>the remaining top after failure A lowers below $Q_{\text{failure,piping,B}}$</td>
</tr>
<tr>
<td>3</td>
<td>Dike ring area</td>
<td>Piping falling</td>
<td>(Physically impossible scenario: failure of B after A will not occur)</td>
</tr>
<tr>
<td>4</td>
<td>Dike ring area</td>
<td>Overtopping</td>
<td>the remaining top after failure A lowers below $Q_{\text{failure,overflow,B}}$</td>
</tr>
<tr>
<td>5</td>
<td>Dike ring area</td>
<td>Piping rising</td>
<td>the remaining top after failure A lowers below $Q_{\text{failure,piping,B}}$</td>
</tr>
<tr>
<td>6</td>
<td>Dike ring area</td>
<td>Piping falling</td>
<td>(Physically impossible scenario: failure of B after A will not occur)</td>
</tr>
<tr>
<td>7</td>
<td>Dike ring area</td>
<td>Overtopping</td>
<td>the remaining top after failure A lowers below $Q_{\text{failure,overflow,B}}$ or if the downstream translation wave reaches $Q_{\text{failure,overflow,B}}$ before B fails.*</td>
</tr>
<tr>
<td>8</td>
<td>Dike ring area</td>
<td>Piping rising</td>
<td>the remaining top after failure A lowers below $Q_{\text{failure,piping,B}}$ or if the downstream translation wave reaches $Q_{\text{failure,piping,B}}$ before B fails.*</td>
</tr>
<tr>
<td>9</td>
<td>Dike ring area</td>
<td>Piping falling</td>
<td>the remaining top after failure A lowers below $Q_{\text{failure,piping,B}}$ or if the downstream translation wave reaches $Q_{\text{failure,piping,B}}$ before B fails.*</td>
</tr>
<tr>
<td>10</td>
<td>Dike ring area</td>
<td>Overtopping</td>
<td>the upstream translation wave after failure B reaches $Q_{\text{failure,overflow,A}}$ before A fails</td>
</tr>
<tr>
<td>11</td>
<td>Dike ring area</td>
<td>Piping rising</td>
<td>the upstream translation wave after failure B reaches $Q_{\text{failure,piping,A}}$ before A fails</td>
</tr>
<tr>
<td>12</td>
<td>Dike ring area</td>
<td>Piping falling</td>
<td>the upstream translation wave after failure B reaches $Q_{\text{failure,piping,A}}$ before A fails</td>
</tr>
<tr>
<td>13</td>
<td>Dike ring area</td>
<td>Overtopping</td>
<td>the upstream translation wave after failure B reaches $Q_{\text{failure,overflow,A}}$ before A fails</td>
</tr>
<tr>
<td>14</td>
<td>Dike ring area</td>
<td>Piping rising</td>
<td>the upstream translation wave after failure B reaches $Q_{\text{failure,piping,A}}$ before A fails</td>
</tr>
<tr>
<td>15</td>
<td>Dike ring area</td>
<td>Piping falling</td>
<td>the upstream translation wave after failure B reaches $Q_{\text{failure,piping,A}}$ before A fails</td>
</tr>
<tr>
<td>16</td>
<td>Dike ring area</td>
<td>Overtopping</td>
<td>(Physically impossible scenario: failure of A after B will not occur)</td>
</tr>
<tr>
<td>17</td>
<td>Dike ring area</td>
<td>Piping rising</td>
<td>(Physically impossible scenario: failure of A after B will not occur)</td>
</tr>
<tr>
<td>18</td>
<td>Dike ring area</td>
<td>Piping falling</td>
<td>the upstream translation wave after failure B reaches $Q_{\text{failure,piping,A}}$ before A fails</td>
</tr>
</tbody>
</table>

*Piping rising/falling means: piping during rising/falling water level

32 $18=2\times3\times3$: 2 dike ring areas and 3 failure mechanisms (overflow/overtopping, piping during rising, and piping during falling water level) for each dike ring area
For example: if in a certain run in a Monte Carlo Simulation scenario 1 (from Table 5.1) occurs and the second failure is indeed prevented, then in that run only A fails due to overtopping. These scenarios are also displayed in the output of the model (appendix N 'Model tests'). More details of the prevention of the failure of each scenario are to be found in appendix L.

Scenario 3, 6, 16 & 17 from Table 5.1 are physically impossible, which is also explained in appendix L.

5.2 **SYSTEM CONFIGURATION II - SHORTCUT BETWEEN DIKE RING AREAS**

5.2.1 **DESCRIPTION**

System configuration II (Figure 5.2) represents a system consisting of one river and two dike ring areas. Both dike ring areas are situated on the northern side of the river. A separating dike separates the two dike ring areas. Breach growth in this dike occurs when water flows from dike ring area A over the dike into B. The dimensions of the dike ring areas as well as the geometry and discharge statistics of the river are the same as in System configuration I, except for the height of the separating dike, which will be varied in the different calculations (explained next).

![Figure 5.2 System configuration II](image)

**Determination dimensions dike ring areas**

The difference between System configuration II and System configuration I is, that in System configuration II B can fail through A if A overflows. The volume of inundation in proportion to the storage volume of A is therefore important. In a highly detailed model of an existing system of rivers and dike ring areas (e.g. the Sobek model used in Picaso [2001]) this proportion is automatically accounted for, as the dimensions of the dike ring areas are known. In this simplified model however, dimensions have to be chosen. The dimension of the dike ring areas are roughly based on dike ring area 43 (Betuwe en Tieler-, en Culemborgerwaarden) as this dike ring area has already been subject of a detailed investigation (Picaso [2001]). In this investigation dike ring area 43 seems to overflow in every failure scenario. This corresponds to experiences in the past, which show that dike ring area 16 (Alblasserwaard/Vijfhererenlanden) ‘frequently’ flooded through 43. In this calculation it is decided to vary the storage volume of dike ring area A in order to investigate the influence of the storage volume of A on the safety of B. The variation of the storage volume is realized by changing the height of the separating dike. At given length and width of A, the storage volume is totally determined by that height. The volumes and corresponding heights of the separating dike are shown in Table 5.2.

<table>
<thead>
<tr>
<th>Height separating dike* [m]</th>
<th>Storage volume <em>[10^9 m^3]</em></th>
</tr>
</thead>
<tbody>
<tr>
<td>6.0</td>
<td>7.57</td>
</tr>
<tr>
<td>6.1</td>
<td>7.81</td>
</tr>
<tr>
<td>6.2</td>
<td>8.05</td>
</tr>
<tr>
<td>6.3</td>
<td>8.29</td>
</tr>
</tbody>
</table>

*Compared to land level in dike ring area (+3 m if compared to bottom of river)
The storage volumes (in increasing order) correspond with the mean volumes of inundation in case A is designed (isolated assessment) with return periods of (500, 1000, 1500 and 2000 year).

The separating dike in system configuration II is always lower than the adjacent river dike.

The maximum damage is always reached when the volume of inundation equals the storage volume corresponding with a height of the separating dike of 6.0 m, irrespective any later elevation of that dike. However, in the determination of a possible overflow of A, the volume of inundation is compared to the actual storage volume of A. So maximum damage of A does not automatically coincides with overflow of A.

5.2.2 Analysis

The interaction through the river is the same as in system configuration I (paragraph 5.1.2). An extra scenario is the situation that B can fail through A. Two conditions have to be met before this scenario can occur:

1. A fails and;
2. Volume of inundation in A > storage volume of A.

In this System configuration the separating dike only fails due to overflow. It is further assumed that if B fails through A, the damage factor (paragraph 4.8) of B is 1: all the water of A will flow into B. Note that in this case indeed holds \( D_{AB} = D_A + D_B \) (see also paragraph 3.3.2 on page 17).

Theoretically, it might occur that B will fail both from the river and through A. In that case the damage factor of B is also 1. When B overflows (i.e. the volume of inundation is larger than its storage volume), the water flows out of the system without inflicting any more damage.
5.3 SYSTEM CONFIGURATION III - SHORTCUT BETWEEN RIVERS

5.3.1 DESCRIPTION

System configuration III (Figure 5.3) represents a system consisting of two rivers and two dike ring areas. In this system configuration the possible points of failure are labeled (A0, A1, B2) for the sake of clarity. The geometry of both rivers is the same, only the statistics of the discharge differ: the discharge that occurs once in 1250 year (design discharge) is 10667 m$^3$/s (River N) or 9020 m$^3$/s (River S) (see also paragraph 4.3.1). The discharges of both rivers are completely correlated, which means that discharge waves always occur simultaneously in both rivers. This assumption is based on the Dutch rivers Maas and Waal: their catchment areas are adjacent for several hundreds of kilometers and historical high water data show simultaneous occurrence of peak discharges in both rivers (appendix P). The complete correlation is applied in the Monte Carlo Simulation by multiplying the two design discharges by the same discharge factor in each run (paragraph 4.3.1).

Dike ring area A has got two possible points of failure, one on each river. If both points of failure of A are designed on the same probability of flooding (e.g. 1/1250 per year), then the dikes along river S are approximately one meter lower than the dikes along river N (see also Figure 3.3 on page 10, System configuration III). This situation is comparable to dike ring area 41 (Land van Maas en Waal, appendix A) in The Netherlands. If the sum of the volumes of inundation from river N (northern side) and river S (southern side) is larger than the storage volume of the dike ring area A, then the water will flow from river N, through dike ring area A into river S. Due to the correlation in river discharges there are high discharges in river S at that moment as well. As river S is smaller than the river N, river S will not be able to cope with the extra discharges resulting in dike breaches in dike ring area B. Going back to the situation in The Netherlands, the average discharge of the river Maas is about 1/6 of the average discharge of the river Waal. The shortcut between Waal and Maas already occurred in the past, Figure 3.4, page 12. In this System configuration it is assumed that from the inside A1 only fails due to overflow.

5.3.2 ANALYSIS

The interaction between A and B through river S is the same as in System configuration I and II. An extra scenario is the situation that B fails through A (in other words: a shortcut between the two rivers is induced). Two conditions have to be met before this scenario can occur:
1. A fails via river N and;
2. Volume of inundation in A > storage volume of A.
In reality a third condition should come into effect: the extra volume in river S leads to failure of B. In this System configuration it is assumed that the probability that this third condition occurs is 1: overflow of A automatically leads to failure of B. The volume of inundation of A is now the sum of the volumes of inundation from both river N and S. So overflow of A (and a resulting shortcut between the two rivers) can be induced in two ways:
1. Failure of A from river N;
2. Failure of A from river N and S.

If A only fails from river S and overflows, it is assumed that the inundated water will just flow back to the river S without subsequent inundation of B. Filling of A will take several days and during that period the water level in the river S will have been lowered considerably because of the failure of A (Figure 4.10, page 31).

The two possible shortcuts between the two rivers are displayed in Figure 5.4. The above-discussed possibility (which does not result in a shortcut as explained) is also shown for the sake of clarity.

An important difference with System configuration II is, that in this System configuration the storage volume of A is not determined by the height of the separating dike, but by the southern dikes of A (along river S): the higher these dikes, the larger the storage volume.

In this system configuration it is (again) assumed that if B fails through A, the damage factor (paragraph 4.8) of B is 1. Note that in this case indeed holds \( D_{AB} = D_A + D_B \) (see also paragraph 3.3.2 on page 17).

If B overflows, the water flows out of the system without inflicting any more damage.
6 CALCULATIONS AND RESULTS

In this chapter the performed calculations are described and the results are presented and analysed. In paragraph 6.1 a general description of the calculation is given. In paragraph 6.2 to 6.4 the calculations and results for system configuration I to III are discussed.

6.1 INTRODUCTION

The intentions of the calculations in this chapter are based on the formulated objectives of this project (paragraph 3.5), which says a.o.:

“*A comparison will be made of probabilities and risk of flooding of different dike ring areas calculated according to the isolated and integrated risk assessment. This will be carried out for three different system configurations of interaction between dike ring areas.*”

Draft of the calculations and analyses

The draft of the analyses is based on Figure 3.6 on page 14 in paragraph 3.3: the system configurations will be assessed by means of an isolated (interaction omitted) and an integrated risk assessment (interaction incorporated). Both assessments will be applied on the total system and on a single dike ring area. The cases which will be studied are shown in Table 6.1.

Table 6.1 Cases studied in this chapter

<table>
<thead>
<tr>
<th>System configuration</th>
<th>Hydraulic system</th>
<th>Paragraph</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td></td>
<td>6.2</td>
</tr>
<tr>
<td>II</td>
<td></td>
<td>6.3</td>
</tr>
<tr>
<td>III</td>
<td></td>
<td>6.4</td>
</tr>
</tbody>
</table>

Next to these given systems, the effects of various dike improvements are investigated in these paragraphs as well.
Presentation of the results
The results of the calculations will (a.o.) be presented in so-called ‘exceedance’- and ‘density’-curves. For both curves a fictitious graph is shown in Figure 6.1.

The exceedance-curve shows the inflicted damage on the horizontal axis and the probability of exceedance of that damage on the vertical axis (generally on a logarithmic scale). The area underneath the curve equals the risk. The probability of failure in this curve equals the probability of exceedance of a damage of zero and is therefore represented by the point of intersection of the curve with the vertical axis.

The density-curve shows the inflicted damage on the horizontal axis and the probability density of that damage on the vertical axis. The area underneath the curve equals 1. So if the probability of the occurrence of damage in general is Pf, the probability of the occurrence of no damage is 1-Pf, which is displayed in the graph by a peak with an area of 1-Pf at a damage of zero (the so-called Dirac-function). The Dirac-function in the density-curve of Figure 6.1 is omitted in the curves in the rest of this chapter for the sake of convenience. These curves therefore only show the probability density of the damage under the condition that failure has occurred and the area underneath these curves therefore represent that probability of flooding.

As the risk in this context is defined as probability of failure * damage, the risk can change by a change of either the probability of failure, or the inflicted damage, or both. The black arrows in Figure 6.1 show what happens to both curves in case of a decrease of the probability of failure, the gray arrows show what happens to both curves in case of a decrease of the inflicted damage. A combination of both may occur as well.

Input
As explained the levels of protection of the dike ring areas are being changed in the different calculations which will be performed. The various safety levels are obtained by changing the strength parameters of the dike: the critical overtopping height (Hcr,o) and the critical piping height (Hcr,p).

In appendix S the values of these parameters are shown for different level of protection (return periods in the isolated assessment), for the different rivers (river N and river S) and for different positions along those rivers (0 km and 65 km).
6.2 SYSTEM CONFIGURATION I - INTERACTION THROUGH A RIVER

6.2.1 SAFETY OF A GIVEN SYSTEM

The following case is being analysed (Figure 6.2): both dike ring areas are designed with a probability of failure of 1/1250 per year according to the isolated assessment. The distance between the points of failure is 65 km along the river and the maximum potential damage for dike ring area A is € 27*10^9 and for B € 54*10^9. The results are shown in Table 6.2.

It appears from Table 6.2 that both dike ring areas benefit from the integrated assessment considering both the probability and risk of flooding. Apparently the downstream dike ring area B takes the most advantage: the mechanisms, which could prevent the second failure (appendix I), are stronger in the downstream direction (lowering of the discharge wave and translation waves) than in the upstream direction (only translation waves). In addition, the velocity of the upstream translation waves are smaller than the velocity of the downstream translation waves (paragraph 4.7).

<table>
<thead>
<tr>
<th>Dike ring area</th>
<th>Return period [Year]</th>
<th>Risk [Mln €/Year]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Isolated</td>
<td>Integrated</td>
</tr>
<tr>
<td>A</td>
<td>1250</td>
<td>1679</td>
</tr>
<tr>
<td>B</td>
<td>1250</td>
<td>1974</td>
</tr>
<tr>
<td>Total system</td>
<td>625</td>
<td>980</td>
</tr>
</tbody>
</table>

In the following the results for the total system and for each single dike ring area are presented in more detail.
Results: total system (Figure 6.3)

Probability of flooding
It is to be seen that the reduction of the risk in the integrated assessment compared to the isolated assessment is mainly caused by a reduction of the probability of flooding of the total system: \( P(A \cup B) \). This is caused by the fact that the probability that A and B both fail \( P(A \cap B) \) can be determined in the integrated assessment (which has also been recognized in paragraph 3.3.2 on page 17).

Inflicted damage
However, a small reduction of the inflicted damage in the integrated assessment can also be perceived: the encircled parts in the curves show the cases in which B fails despite failure (and even complete inundation: maximum damage) of A: \( P(A \cap B) \). The inflicted damage to dike ring area B in these cases is relatively low, as most of the river water has already flown into dike ring area A. (appendix J: “Volume of inundation in case of second failure”). In the isolated assessment, B would have had a much larger damage (around € 5.4E+10). This interaction between inflicted damages has already been recognized in paragraph 3.3.2 on page 17: \( D_{AB} < D_A + D_B \). It appears from Figure 6.3 that in this system configuration (in which only interaction through the river plays a role) the effects on the risk of the interaction between inflicted damages is only minor compared to the effects of the interaction between probabilities of flooding.

![Figure 6.3 Exceedance (top) and density (bottom) curves for total system.](image-url)
Results: single dike ring area (Figure 6.4)

Probability of flooding
For each single dike ring area also holds that the reduction of the risk in the integrated assessment is mainly caused by a reduction of the probability of flooding. It is clear from this figure that B benefits the most from the integrated assessment, as the reduction of the probability of flooding of B: \( P(B \cap \overline{A}) + P(A \cap B) \) is larger than the reduction of the probability of flooding of A: \( P(A \cap \overline{B}) + P(A \cap B) \).

Inflicted damage
In Figure 6.4 the encircled spots from Figure 6.3 (page 49), which represent some of the cases in which A and B both fail, are again shown.

---

It has to be noted that there is hardly any difference between the density-curves for each single dike ring area (Figure 6.4, bottom) and the density curves for the total system (Figure 6.3 on page 49, bottom). In the following of this chapter only the exceedance- and density-curves for the total system are presented, as exceedance- en density-curves for each single dike ring area do not really add any new information.
INTERMEZZO: VARIED DISTANCE BETWEEN POINTS OF FAILURE

In the preceding calculation as well as in all the coming calculations, the distance between the possible points of failure has been fixed on 65 km. Now it is investigated what happens to the probabilities and risks of flooding of the dike ring areas for different distances: 0 km, 65 km and 130 km, shown in Figure 6.5. The results are presented in Table 6.3 (probabilities and risks of flooding) and Figure 6.6 (exceedance- and density-curves).

Table 6.3 Return periods [Year]

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>System</td>
<td>985</td>
<td>40.2</td>
<td>960</td>
<td>39.5</td>
<td>942</td>
<td>36.5</td>
</tr>
</tbody>
</table>

Figure 6.5 Varied distance between possible points of failure: 0 km (left), 65 km (centre) and 130 km (right)

Figure 6.6 Exceedance- and density-curves for total system
Probability of flooding

It appears that the probability of flooding of B is not influenced by its position compared to A: \( P(B \cap \overline{A}) + P(A \cap \overline{B}) \) remains constant in Figure 6.6: even though \( P(B \cap \overline{A}) \) decreases (with increasing distance), \( P(A \cap B) \) increases (with increasing distance).

However, the position of B does influence the correlation probability of flooding of A. As the distance between A and B becomes larger, the probability of flooding of A \( P(A \cap \overline{B}) + P(A \cap \overline{B}) \) increases: the larger the distance, the less frequently B is able to prevent failure of A (increase \( P(A \cap \overline{B}) \)) and failure of both A and B will occur more often (increase \( P(A \cap \overline{B}) \)).

Inflicted damage

For the same reason both the risk of A and B are influenced by the distance: the larger the distance, the larger the risk of A and the smaller the risk of B. The reduction of the risk of B (during increasing distance) is striking as the probability of flooding of B remains constant. This risk reduction of B is only possible if smaller damages of B occur more often as the distance increases, which is indeed the case: \( P(A \cap \overline{B}) \) in Figure 6.6. The inflicted damage to dike ring area B in these cases is relatively low, as most of the river water has already flown into dike ring area A. (appendix J: “Volume of inundation in case of second failure”). It has to be noticed that \( P(A \cap \overline{B}) = 0 \) in the case that the two dike ring areas are directly opposite to each other (distance is 0 km).

6.2.2 DIKE IMPROVEMENT

In the preceding paragraph the effects of the incorporation of interactions between dike ring areas on a given hydraulic system have been analysed. One step further is the investigation of the effects of measures in the system, such as dike improvement, on the safety of dike ring areas in an integrated approach.

Assume the following system configuration (Figure 6.7) of two dike ring areas (A and B) and two failure scenarios (1 and 2). Dike improvement of a dike ring area has effects on the probability of occurrence of these failure scenarios. These effects are schematised presented as follows (↑ means: increasing probability of occurrence; ↓ means: decreasing probability of occurrence):

- Dike improvement of A: 1 ↓ 2 ↑
- Dike improvement of B: 1 ↑ 2 ↓

Starting-point for the dike improvement in this paragraph is the case of Figure 6.2 on page 48: alternately the dikes of A and B are being improved in such a way a return period of 2000 is obtained (in the isolated assessment), which yields the following three cases (Figure 6.8). The results are presented in Table 6.4 and Figure 6.9 on page 54.
The following can be concluded from Table 6.4 and Figure 6.9 on page 54:

1. Improvement of the level of protection of a certain dike ring area has got positive effects locally (reduction probability and risk of flooding) but has negative effects elsewhere in the system;

2. A dike ring area approaches its ‘isolated’ return period (1/1250 per year) as the other dike ring area becomes safer (e.g. 1/2000 per year). This is logical, as the isolated assessment implicitly presumes infinitely strong dikes in other areas;

3. The probability of flooding of the total system is equal in both improvement scenarios. However, the distribution of the probability of flooding over the two dike ring areas differs. Therefore the risk of the total system differs as well in the two improvement scenarios. If it is assumed that both measures of improvement are equally expensive, improvement of B is the most favorable, as it yields the largest reduction of the risk of the total system;

4. The downstream effect of the interaction between the dike ring areas is larger than the upstream effect: an increase of the level of protection of A has more effect on B (1974 compared to 1518 year), than the same increase of safety of B has on A (1679 compared to 1448 year);

Table 6.4 Return periods and risks for dike ring areas A and B and for total system (integrated assessment)

<table>
<thead>
<tr>
<th>Dike ring area</th>
<th>Starting point</th>
<th>Improvement of A</th>
<th>Improvement of B</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1679</td>
<td>16.4</td>
<td>3901</td>
</tr>
<tr>
<td>B</td>
<td>1974</td>
<td>23.1</td>
<td>1518</td>
</tr>
<tr>
<td>System</td>
<td>980</td>
<td>39.5</td>
<td>1170</td>
</tr>
</tbody>
</table>
More scenarios of dike improvement are to be found in appendix P from which is to be concluded that the effects of the incorporation of the interaction is the most distinct when the ‘isolated’ return periods of the dike ring areas are wide apart, in other words: if one dike ring area is significantly safer than the other one. The safest dike ring area benefits to a large extent from the other dike ring area which is less safe.
6.2.3  EMERGENCY STORAGE AREA

**General**
In this paragraph a very simple investigation of the effectiveness of emergency storage areas (paragraph 2.2.3 ‘Flood storage’) is made. The use of such an area should prevent failure of other downstream situated dike ring areas. As was shown in the preceding, a normal dike ring area can also prevent failure of a more downstream situated dike ring area. The advantage of an emergency storage area is that only the top of the discharge wave is stored, so the damage to that area will be significantly smaller than in case of an uncontrolled flooding (resulting in the development of a large breach and consequently a large volume of inundation). It is assumed that if just the peak of the discharge wave is being stored in the emergency storage area (‘peak shaving’, paragraph 4.5.2), failure of the downstream dike ring area will (just) be prevented.

**Design of the inlet work: weir**
In this case dike ring area A (Figure 6.2 on page 48) is designed as an emergency storage area. The inlet work of the emergency storage area is assumed to be a fixed concrete weir (instead of a normal dike of clay and sand). The height of the threshold of the weir determines if and when the storage area comes into effect. The higher the threshold, the lower the probability that the emergency storage area is flooded, but the higher the probability that failure of a downstream situated dike ring area will not be prevented. The determination of the height of the threshold of the weir is therefore very important. In this analysis the height of the threshold is designed according to the isolated assessment on three different probabilities of exceedance: 1/1250, 1/1000 and 1/750 per year; the higher the probability of exceedance, the lower the threshold. It is assumed that the width of the weir is always large enough to let all the water in above the critical height. It is assumed that in every situation the emergency storage area functions as it should, and that it does not fail due to piping. The height of the threshold has got a standard deviation of 0.1 m, which is smaller than the standard deviation in case of a normal dike (0.3 m, paragraph 4.4.1). This is due to the (probably) accurate construction of the (concrete) threshold and the (probably) accurate determination of the water height-discharge relation on the spot. In addition the concrete construction will not fail due to wave overtopping.

**Results and analyses**
The results are presented in Table 6.5 and Figure 6.10. For the sake of comparison the starting-point situation (Figure 6.2 on page 48) is also presented. The main conclusion is that the application of an emergency storage area only reduces the probability and risk of flooding of B if the probability of flooding of the emergency storage area (A) is high enough: i.e. significantly higher than in the case that A is a normal dike ring area (±1/1000 per year compared to 1/1679 per year).

<table>
<thead>
<tr>
<th>Dike ring area</th>
<th>ESA 1250</th>
<th>ESA 1000</th>
<th>ESA 750</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1679</td>
<td>16.4</td>
<td>1701</td>
</tr>
<tr>
<td>B</td>
<td>1974</td>
<td>23.1</td>
<td>1730</td>
</tr>
<tr>
<td>System</td>
<td>980</td>
<td>39.5</td>
<td>996</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Figure 6.10 Exceedance- and density-curves of total system

It appears from Figure 6.10 that the damage of dike ring area A reduces significantly if A is designed as an emergency storage area: the volume of inundation in case of flooding of an emergency storage area (peak shaving) is much less than the volume of inundation in case of an uncontrolled dike breach (compare Figure 4.13 to Figure 4.14 on page 33). It is also interesting to see what happens to dike ring area B: in the case of the ‘1/1250’ design of the threshold (which was also the ‘isolated’ probability of failure in case of A being a normal dike ring area), the probability of flooding (and therefore the risk) of B even becomes larger, instead of smaller. This is caused by the fact that the uncertainty of the strength parameters of A is smaller in case of an emergency storage (0.1 m) than in the case of A being a normal dike ring area (0.3 m): prevention of failure of B by ‘failure’ of A occurs less frequently. Only in case of the ‘1/750’ design, the emergency storage area reduces the risk of B compared to the case of A being a normal dike ring area. In this case A is inundated more frequently compared to the ‘normal dike ring area’ design. If the probability density curve of B is investigated in a little more detail, it appears that in the case of A being an emergency storage area, damages higher than the maximum damage of B (€ 5.4*10^9) can occur (encircled in Figure 6.10). This part of the curve represents the cases in which application of the emergency storage area is not able to prevent failure (and overflow) of B.

The conclusion is that in these calculations the risk of the total system in the cases of A being an emergency storage area is always smaller than in the case of A being a normal dike ring area. However, risk reduction of B (the downstream dike ring area) only occurs if the threshold is rather low.
6.2.4 MAIN CONCLUSIONS

The following main conclusions can be drawn considering the preceding analysis of system configuration I:

1. Both dike ring areas benefit from the incorporation of the interaction in the assessment of their probabilities and risks of flooding: both are reduced;
2. The downstream situated dike ring area benefits the most from the incorporation of the interaction;
3. The effects of the incorporation of the interaction on the probabilities and risks of flooding are the most significant in case of a significant initial (according to the isolated assessment) distinction in levels of protection (e.g. one dike ring area is being designed on a probability of flooding of 1250 years, the other on 2000 years, which is a common situation in The Netherlands (appendix A);
4. Local dike improvement has got local positive effects (decrease of probability and risk of flooding), but might induce negative effects elsewhere in the system (increase of probability and risk of flooding). The effects of local dike improvement should therefore be considered integrally (considering the total system) in order to enable a correct cost-benefit calculation;
5. The design (e.g. height of the threshold or the operation of a gated inlet work) of an emergency storage area should be performed thoroughly, otherwise the effects might even turn out negative for the downstream situated dike ring areas.
6.3 SYSTEM CONFIGURATION II - SHORTCUT BETWEEN DIKE RING AREAS

6.3.1 SAFETY OF A GIVEN SYSTEM

The following case is being analysed (Figure 6.11): the dike ring areas are the same as in paragraph 6.2.1, Figure 6.2 on page 48, only the configuration differs (both dike ring areas on the same side of the river), resulting in the introduction of the possibility of a shortcut between the dike ring areas. The probabilities and risks of flooding are calculated according to the isolated and integrated risk assessment. The results are presented in Table 6.6 and in Figure 6.13 on page 59.

![Figure 6.11 System configuration II](image)

Table 6.6 Results System configuration II, situation 1

<table>
<thead>
<tr>
<th>System Level</th>
<th>Return period [Year]</th>
<th>Risk [Mln €/Year]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Isolated</td>
<td>Integrated</td>
</tr>
<tr>
<td>A</td>
<td>1250</td>
<td>1679</td>
</tr>
<tr>
<td>B</td>
<td>1250</td>
<td>1237</td>
</tr>
<tr>
<td>System</td>
<td>625</td>
<td>980</td>
</tr>
</tbody>
</table>

It appears from Table 6.6 that B hardly benefits from the interaction as neither the probability of flooding (or return period), nor the risk of B changes significantly if the integrated assessment is performed. The positive interaction (interaction through the river) is compensated by the negative interaction (shortcut between dike ring areas). Only B is influenced by this negative interaction; the probability of flooding of the system is not influenced as is explained in Figure 6.12. Only the probability that A and B both fail increases compared to the situation without shortcut. Therefore the risk of the system is larger in the situation including a shortcut, although still less than in the isolated approach.

![Figure 6.12 Failure of the total system with and without shortcut](image)
Probability of flooding
It is to be seen (Figure 6.13) that the probability of flooding of the total system $P(A \cup B)$ in the integrated assessment is still less than in the isolated assessment, however, the probability that A and B both fail $P(A \cap B)$ increases in the integrated assessment due to the possibility of a shortcut. The probability that A and B both fail through the river (encircled part in Figure 6.13) is now almost 0. The probability that only A fails $P(A \cap B)$ clearly decreases, to the detriment of the probability that both A and B fail $P(A \cap B)$, which increases.

Inflicted damage
It is to be seen (Figure 6.13) that the probability of occurrence of ‘lower’ damages (around € 27E+9 and € 54E+9) decreases and the probability of occurrence of ‘higher’ damages (around € 8.1E+9) increases in the integrated assessment. In this particular situation the risk of B and of the total system in the integrated assessment still stays below the risk of B and of the total system in the isolated assessment. The isolated assessment is therefore still a conservative approach. However, if the levels of protection of the dike ring areas as well as their potential damages are changed, the isolated assessment might not remain a conservative approach any longer as will be shown in the following paragraph.

Figure 6.13 Exceedance (top) and density (bottom) curves for total system
6 Calculations and results

6.3.2 DIKE IMPROVEMENT

In the preceding paragraph the effects of the incorporation of interactions between dike ring areas on a given hydraulic system have been analysed. One step further is the investigation of the effects of measures in the system, such as dike improvement, on the safety of dike ring areas in an integral approach.

Assume the following system configuration (Figure 6.14) of two dike ring areas (A and B) and three failure scenarios (1, 2 and 3). Dike improvement of a dike along the river or of the separating dike has effects on the probability of occurrence of these failure scenarios. These effects are schematised presented as follows (∆ means: increasing probability of occurrence, ↓ means: decreasing probability of occurrence, — means: no change):

Dike improvement of A:  1 ↓  2 ↑  3 ↓
Dike improvement of B:  1 ↑  2 ↓  3 ↑
Improvement separating dike:  1 —  2 —  3 ↓

Dike improvement A and B

Starting-point for the dike improvement in this paragraph is the case of Figure 6.11 on page 58: alternately the dikes of A and B are being improved in such a way a return period of 2000 is obtained (in the isolated assessment), which yields the following three cases (Figure 6.15). The results are presented in Table 6.7. and Figure 6.16 on page 61.

1. The most striking difference compared to paragraph 6.2.2 (dike improvement in system configuration I) is, that B does not seem to be influenced by the improvement of A at all (which was the case in paragraph 6.3.1 as well). The return period and risk of B remain the same after all. However, a shift in failure scenarios of B takes place: the probability that B fails through A (shortcut: $P(A \cap B)$ in Figure 6.16) decreases as the probability that B fails through the river ($P(B \cap \overline{A})$ increases. Apparently the decrease of $P(A \cap B)$ balances the increase of $P(B \cap \overline{A})$. The probability that A and B both fail through the river (encircled part in Figure 6.16) is now almost 0;

2. If both improvement measures would have been equally expensive, improvement of A s the better option as the reduction of the risk of the total system is the largest, in contrast to system configuration I (paragraph 6.2.2 on page 53) where improvement of B was the better option;
3. It still holds that improvement of the level of protection of a certain dike ring area has got positive effects locally, but can have has negative effects elsewhere in the system;

4. In case of improvement of B, the probability of flooding of B in the integrated assessment (1/1701 per year) is significantly larger than the probability of flooding of B in the isolated assessment (1/2000 per year), which means that in this case the isolated assessment is no longer a conservative approach.

Table 6.7 Return periods and risks for dike ring areas A and B and for total system (integrated assessment)

<table>
<thead>
<tr>
<th></th>
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<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1679</td>
<td>16.4</td>
<td>3843</td>
<td>6.9</td>
<td>1408</td>
<td>18.7</td>
</tr>
<tr>
<td>B</td>
<td>1237</td>
<td>42.0</td>
<td>1223</td>
<td>42.0</td>
<td>1701</td>
<td>31.1</td>
</tr>
<tr>
<td>System</td>
<td>980</td>
<td>58.4</td>
<td>1170</td>
<td>48.9</td>
<td>1170</td>
<td>50.0</td>
</tr>
</tbody>
</table>

Figure 6.16 Exceedance (top) and density (bottom) curves for total system
6 Calculations and results

**Improvement separating dike**
Starting-point for the dike improvement is again the case of Figure 6.11 on page 58. In this case the separating dike is improved from 6.0 m (compared to the land level in the dike ring area) to 6.3 m (this improvement of 0.3 m is in the same order of magnitude as the improvement of the river dikes). The results are presented in Table 6.8 and Figure 6.18 page 62.

![Figure 6.17 Improvement separating dike](image)

**Table 6.8 Return periods and risks for dike ring areas A and B and for total system (integrated assessment)**

<table>
<thead>
<tr>
<th>Dike ring area</th>
<th>Starting point</th>
<th>Improvement separating dike</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Return period</td>
<td>Risk [Mln €/Year]</td>
</tr>
<tr>
<td>A</td>
<td>1679</td>
<td>16.4</td>
</tr>
<tr>
<td>B</td>
<td>1237</td>
<td>42.0</td>
</tr>
<tr>
<td>System</td>
<td>980</td>
<td>58.4</td>
</tr>
</tbody>
</table>

![Figure 6.18 Exceedance (top) and density (bottom) curves for total system](image)
The following can be concluded:

1. It can be concluded that improvement of the separating dike decreases the probability and risk of flooding of B as well as the risk of the total system, which is to be seen in Figure 6.18 from the decrease of $P(A \cap B)$;

2. In the initial situation maximum damage of A also means overflow of A. In the improved situation the storage capacity of A increases, but the maximum damage is still related to the initial storage capacity. So in the improved situation maximum damage of A can occur without overflow of A, which is to be seen in Figure 6.18. The probability of flooding of A: $P(A) = P(A \cap B) + P(A \cap B)$ remains constant;

3. As explained in Figure 6.12 on page 58 the probability of flooding of the total system remains constant as well.

More scenarios of dike improvement are to be found in appendix Q from which is to be concluded that the effects of the incorporation of the interaction is the most distinct when the ‘isolated’ return periods of the dike ring areas are wide apart, in other words: if one dike ring area is significantly safer than the other one. The safest dike ring area benefits to a large extent from the other dike ring area which is less safe.

### 6.3.3 MAIN CONCLUSIONS

In addition to 6.2.4 (main conclusions regarding system configuration I), the following main conclusions can be drawn considering the preceding analysis of system configuration II:

1. Compared to system configuration I (interaction through the river), dike ring area B benefits significantly less from the incorporation of the interaction in system configuration II due to the (negative) shortcut between the dike ring areas. In contrast to system configuration I the isolated assessment in system configuration II is not necessarily a conservative one: In the isolated assessment of the downstream dike ring area B holds:

   $P(B) = P(Z_B < 0)$ \hspace{1cm} (6.1)

   Assume that the storage capacity of A is relatively small, so failure of A automatically yields failure of B. For the probability of failure of B in the integrated assessment now holds:

   $P(B) = P(Z_A < 0 \cup Z_B < 0)$ \hspace{1cm} (6.2)

   As $Z_A$ and $Z_B$ are correlated (the load is the same in each limit state function), the following inequality is in effect:

   $P(Z_A < 0 \cup Z_B < 0) > P(Z_B < 0)$, \hspace{1cm} (6.3)

   which means that B can be significantly less safe than expected according to the isolated assessment.
An analogy is a system of bars (strength of each bar stochastic variable \( R \)) which are loaded by the same load (\( S \), also stochastic variable). Such systems are shown in Figure 6.19. The probability failure of system 2 (comparable to the integrated assessment of dike ring area B) is higher than the probability of failure of system 1 (comparable to the isolated assessment of dike ring area B). In other words: the larger the number of links in a chain, the larger the probability that one of them is too weak;

![Figure 6.19 Analogy: system of bars](image)

2. If a dike ring area is under influence of both positive interaction (interaction through the river) and negative interaction (shortcut between dike ring areas) like dike ring area B in system configuration II, local dike improvement with local positive effects (e.g. on dike ring area A), causes a shift in failure scenarios of dike ring area B. It depends on the features of the system whether the dominating effects of interaction on B after the dike improvement are positive or negative and if the probability and risk of flooding will exceed the values calculated according to the isolated assessment. Especially the storage capacity of the dike ring areas compared to the volume of the discharge wave is important in this issue.

### 6.4 System Configuration III - Shortcut Between Rivers

#### 6.4.1 Safety of a Given System

In this calculation the three possible points of failure of System configuration III are being designed with a probability of failure of 1/1250 per year (Figure 6.5). It has to be noted that A in this system configuration has got two possible points of failure. Due to the uncorrelated strength and correlated load parameters, the probability of failure of A in the isolated assessment will be somewhere between 1/1250 and 2/1250. The three possible points of failure are labeled: A0, A1 and B2. The distance between the points of failure is 65 km and the maximum potential damage for A is € 27*E+9 and for B € 54*E+9. The probabilities of failure and risks are calculated in the isolated and integrated risk assessment. The results are presented in Table 6.9 and clarified on the following pages.

![Figure 6.20 System configuration III](image)

<table>
<thead>
<tr>
<th>System Level</th>
<th>Return period [Year]</th>
<th>Risk [Mln €/Year]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Isolated</td>
<td>Integrated</td>
</tr>
<tr>
<td>A</td>
<td>933</td>
<td>959</td>
</tr>
<tr>
<td>B</td>
<td>1250</td>
<td>1155</td>
</tr>
<tr>
<td>System</td>
<td>534</td>
<td>805</td>
</tr>
</tbody>
</table>
Results: total system (Figure 6.21)
The probability of flooding of the total system is smaller in the integrated assessment than in the isolated assessment. This is due to the fact that both dike ring areas benefit from the interaction through the river (positive interaction). However, the possibility of a shortcut between the rivers in the integrated approach increases the possible inflicted damages considerably. Therefore the risk of flooding of the total system increases in the integrated assessment in contrast to the probability of flooding.

In the density-curve (Figure 6.21, bottom) the separate possibilities $P(A \cap \overline{B}), P(B \cap \overline{A})$ and $P(A \cap B)$ are no longer unambiguously to be distinguished (as e.g. in Figure 6.3 on page 49). This is due to the fact that in this system configuration two rivers can inundate two dike ring areas.
Results: single dike ring areas (Figure 6.22)

It is to be seen in Figure 6.22 that A slightly benefits from the interaction through the river. However, the decrease of the probability of flooding of A is only marginal as only the southern point of failure of A benefits from this interaction. The increase of the probability of failure and risk of B is caused by the possibility of a shortcut between the rivers.

![Figure 6.22 Exceedance (top) and density (bottom) curves for dike ring area A and B](image)

*Figure 6.22 Exceedance (top) and density (bottom) curves for dike ring area A and B*
6.4.2 DIKE IMPROVEMENT

In the preceding paragraph the effects of the incorporation of interactions between dike ring areas on a given hydraulic system have been analysed. One step further is the investigation of the effects of measures in the system, such as dike improvement, on the safety of dike ring areas in an integral approach. This analysis is performed in this paragraph.

Assume the following system configuration (Figure 6.23) of two dike ring areas (A and B) and four failure scenarios (1, 2, 3 and 4). Dike improvement of a dike along the river or of the separating dike has effects on the probability of occurrence of these failure scenarios. These effects are schematised presented as follows (↑ means: increasing probability of occurrence, ↓ means: decreasing probability of occurrence, — means: no change):

![Figure 6.23 Failure scenarios (1, 2, 3 and 4) in system configuration III)](image)

Dike improvement of A0: 1 ↓ 2 — 3 ↓ 4 —
Dike improvement of A1: 1 — 2 ↓ 3 ↓ 4 ↑
Dike improvement of B2: 1 — 2 ↑ 3 ↑ 4 ↓

Starting-point of the dike improvement is Figure 6.20 on page 64 in which all the possible points of failure have got a return period of 1250 year (according to the isolated assessment). Subsequently all possible points of failure are being improved alternately until a return period of 2000 year is reached (Table 6.10). Results in Table 6.11.

Table 6.10 Return periods according to isolated assessment

<table>
<thead>
<tr>
<th>Possible point of failure</th>
<th>Starting-point</th>
<th>Improvement of A0</th>
<th>Improvement of A1</th>
<th>Improvement of B2</th>
</tr>
</thead>
<tbody>
<tr>
<td>A0</td>
<td>1250</td>
<td>2000</td>
<td>1250</td>
<td>1250</td>
</tr>
<tr>
<td>A1</td>
<td>1250</td>
<td>1250</td>
<td>2000</td>
<td>1250</td>
</tr>
<tr>
<td>B2</td>
<td>1250</td>
<td>1250</td>
<td>1250</td>
<td>2000</td>
</tr>
</tbody>
</table>

Table 6.11 Results dike improvements

<table>
<thead>
<tr>
<th>Starting-point</th>
<th>Improvement of A0</th>
<th>Improvement of A1</th>
<th>Improvement of B2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Return period [Year]</td>
<td>Risk [Mln €/Year]</td>
<td>Return period [Year]</td>
</tr>
<tr>
<td>A</td>
<td>959</td>
<td>23.9</td>
<td>1178</td>
</tr>
<tr>
<td>B</td>
<td>1155</td>
<td>38.4</td>
<td>1297</td>
</tr>
<tr>
<td>System</td>
<td>805</td>
<td>62.3</td>
<td>861</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>Return period [Year]</th>
<th>Risk [Mln €/Year]</th>
<th>Return period [Year]</th>
<th>Risk [Mln €/Year]</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1148</td>
<td>19.8</td>
<td>967</td>
<td>24.2</td>
</tr>
<tr>
<td>B</td>
<td>1160</td>
<td>36.6</td>
<td>1582</td>
<td>31.6</td>
</tr>
<tr>
<td>System</td>
<td>889</td>
<td>56.2</td>
<td>924</td>
<td>55.8</td>
</tr>
</tbody>
</table>
Improvement of A0
1. It is to be seen that the return period of A increases as the possibility of inundation through A0 decreases. Therefore the risk of A also decreases;
2. The return period of dike ring area B increases as the possibility of a shortcut between the rivers decreases (failure scenario 3 in Figure 6.23). Therefore the risk of B also decreases;
3. As expected the probability and risk of flooding of the total system also decrease.

Improvement of A1
1. The return period of dike ring area B does hardly change, despite the improvement of A1. The increase of the dike height of ‘A1’ has got two possible effects on the safety of B:
   1. Failure of B through A will occur less frequently (positive);
   2. Failure of B through the river will occur more frequently (negative).

   It appears that the increase of frequency of occurrence of one effect is compensated by the decrease of the other. Therefore the return period of dike ring area B remains more or less constant;
2. However, the risk of B decreases as the failure scenario which inflicts the highest damage to B (shortcut between the rivers) occurs less frequently;
3. As expected the probability and risk of flooding of the total system also decrease.

Improvement of B2
1. Even though the return period of B in the integrated risk assessment increases when its dikes are improved, the increase is not as fast as in the isolated assessment (1582 compared to 2000 year) as the possibility of failure of B through A (shortcut between rivers) still exists. The possibility of failure of B through A even increases: although the dike improvement of B makes B more resistant against direct threats from river S, A will be inundated more frequently through river S (interaction through river) resulting in a higher mean volume of inundation of A (also inundated by river N) which causes a more frequent overflow of A and therefore more inundation of B through A (failure scenario 3 in Figure 6.23).
   As the safety of B does still increase, it can be concluded that the positive effect of the decrease of failure of B though river S overcompensates the negative effect of the increase of failure through A (shortcut between rivers).
2. It appears that the probability of flooding of A stays more or less constant (a very slight increase can be perceived) during increasing safety of B, although one might probably have expected a more significant increase because of the interaction through river S (increase of the possibility of scenario 2 in Figure 6.23). The same holds for the risk of A.
   The following arguments can be adduced:
   > The probability of flooding of A does hardly increase, as the increase of failure of A through river S hardly induces more failures of A, as A in those cases would already have failed through river N;
   > The risk of flooding of A does hardly increase, as the extra inundations of A through river S mainly coincide with inundations through river N, which would already have caused overflow of A (and therefore maximum inflicted damage) without inundation through river S. Enlarging the volume of inundation does no longer increase the inflicted damage.
In a (relative) complex hydraulic system as described in this chapter, dike improvement somewhere in the system has got widespread consequences on both probabilities of failure and risks, which have to be investigated and incorporated in the decision-process. The results of the various discussed dike improvement for the total system are once again presented in Table 6.12.

If it is assumed that the improvement measures are all equally expensive, the improvement of B2 would be the best choice if only the return period of the system is considered. However, if the risk of the system is considered, improvement of A0 is the best choice, as this improvement yields the largest reduction of the risk of the system.

Table 6.12 Results different improvement measures for the total system

<table>
<thead>
<tr>
<th>Improvement</th>
<th>Return period [Year]</th>
<th>Risk [Mln €/Year]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial situation</td>
<td>805</td>
<td>62.3</td>
</tr>
<tr>
<td>Improvement A0</td>
<td>861</td>
<td><strong>52.6</strong></td>
</tr>
<tr>
<td>Improvement A1</td>
<td>889</td>
<td>56.2</td>
</tr>
<tr>
<td>Improvement B2</td>
<td>924</td>
<td>55.8</td>
</tr>
</tbody>
</table>

More scenarios of dike improvement are to be found in appendix R.

6.4.3 MAIN CONCLUSIONS

As in system configuration II, if a dike ring area is the under influence of both positive interaction (interaction through the river) and negative interaction (shortcut between rivers) like dike ring area B in system configuration III, local dike improvement with local positive effects (e.g. on dike ring area A), causes a shift in failure scenarios elsewhere in the system (e.g. dike ring area B). It depends on the features of the system whether the dominating effects of interaction on B after the dike improvement are positive or negative and if the probability and risk of flooding will exceed the values calculated according to the isolated assessment.
7 CONCLUSIONS AND RECOMMENDATIONS

In this chapter the main conclusions are presented (paragraph 7.1) as well as the recommendations (paragraph 7.2).

7.1 CONCLUSIONS

In this project the effects of the incorporation of the interaction between dike ring areas on the probability of failure and risk of these dike ring areas are determined. In this paragraph the main conclusions are presented.

As a result of this research project the following conclusions can be drawn:

1. In a system configuration in which only interaction through a river plays a role, the integrated assessment has got a significantly positive influence on the calculated risk of the dike ring areas. The isolated assessment is therefore a conservative approach;

2. However, in a more complex system in which new ways of failure are being introduced (shortcut between dike ring areas and rivers) the results of the integrated risk assessment might become negative, which means higher risk in the integrated risk assessment compared to the isolated risk assessment. Whether the isolated assessment does indeed underestimate the risks, depends on the features of the system (a.o. dike heights, storage volume of dike ring areas);

3. The influence of the integrated risk assessment on the risk of dike ring areas is mainly caused by the influence on the probability of failure., although some influence on the damage does exist as well, depending on the configuration of the system;

4. A measure in the system with local positive effects (e.g. dike improvement), can lead to negative effects (increasing probability of failure and risk) elsewhere, although the probability of failure of the total system will always decrease. The risk of the system however can increase, because local risk-reduction can be overcompensated by an increase of the risk elsewhere in the system. This means that investments, originally meant to reduce the risk of the system, might increase the risk of the system.

Overall it can be concluded that both for a correct assessment of the current probabilities of failure and risks of dike ring areas as well as of the effects of possible future measures (dike improvement, emergency storage areas) the integrated risk assessment is required.
7.2 **Recommendations**

As a result of this research project the following recommendations are made:

**Recommendations regarding calculation results**

1. Despite the fact that the analysed system configurations in this project are very simple compared to a realistic system like the Dutch upper river system, it is, regarding the results, recommended to find out how to incorporate the integrated risk assessment in the current risk analyses of the dike ring area in The Netherlands;

2. As it is expected that (1) is not immediately possible regarding the current state of computer calculation capacity and the integration of the required detailed models, at least negative scenarios of system behaviour (shortcuts between dike ring areas and rivers) in The Netherlands should be analysed in order to determine whether their occurrence is of significant influence on the safety of other dike ring area. This could even be performed deterministically if necessary;

3. An investigation of the way in which a (complex) hydraulic system of dike ring areas and rivers can be economically optimized.

**Recommendations regarding calculation method**

1. Increase of the calculation capacity (by the application of e.g. parallel processing) is desirable in order to enable a more detailed model (2-dimensional modeling of the river, more failure mechanisms, more water defence structures, breach development as a function of actual occurring hydraulic loads etc.);

2. Next to the increase of calculation capacity, the efficiency of the Monte Carlo Simulation can be increased as well by means of the application of Importance Sampling (which reduces the necessary number of runs);

3. An investigation of the influence of more possible points of failure per dike ring area, an investigation of the sensitivity of the model to other shapes of the discharge wave and breach growth parameters and an investigation of the sensitivity of the model to variations in the standard deviations of the dike height;

4. The introduction of human intervention during inundation: e.g. deliberate destruction of a river dike of an already inundated dike ring area, which enables the inundated water to flow back to the river, places of sand bags etc.
LITERATURE

CPB, Werkplan Kosten-Batenanalyse Ruimte voor de rivier, 2002
CUR, Kansen in de civiele techniek, Deel I: Probabilistisch ontwerpen in theorie, Gouda, 1997
d’Angremond, pompen of verzijken, Delft, 2001
Dantzig, van, D. en J. Kriens, Rapport Deltacommissie Deel III, bijdrage II.2: Het economisch beslissingsprobleem inzake de beveiliging van Nederland tegen stormvloeden, Gravenhage, 1960
Delft Cluster 02.01.01, Systemwerking, 2003
DWW, et al., Hydraulische randvoorwaarden 200, voor het toetsen van primaire waterkeringen, Dordrecht, 2001
Huisman, P. en J. Wessel, Waterrecht, Delft, 2000
Jager, de, F.G.J., Beschermen tegen overstromen, onderzoek naar een veiligheidsbeleving op basis van risico’s, Delft, 1998
Jong, de, J. et al., Integraal waterbeheer, Delft, 1999
Jonkman, S.N., Overstromingsrisico’s: een onderzoek naar de toepasbaarheid van risicomening, Delft, 2001
Jonkman, S.N., System behaviour: societal aspects and decision making, 2003
Luteijn, D., Gecontroleerd overstroomen, advies van de commissie noodoverloopgebieden, 2002
Manen, van, S.E., PICASO deel I t/m VI, 2001
Middelkoop, H., Twee rivieren, Rijn en Maas in Nederland, Riza rapport, Arnhem, 1998
Mierlo, van, Th., e.a., Effects of system behaviour on flood risk, Delft 2003
Ministerie van Verkeer en Waterstaat, Ruimte voor de rivier, discussienuit, 2000
Plate, E.J., Flood management as part of sustainable development, Karlsruhe. On Int. Symposium on River Flood Defence, Kassel, 2000
RIKZ, et al, De keerzijde van ons klimaat.
Silva, W. E.a., Ruimte voor Rijntakken, wat het onderzoek ons heeft geleerd, Wateringen, 2000
Steenbergen, H.M.G.M.en A.C.W.M. Vrouwenvelde, Gebruikershandleiding PC-RING, Delft, 2001
TAW, Technisch rapport waterkerende grondconstructies, 2001
TAW, Leidraad Zee- en Meerdijken basissrapport, Delft, 1999
TAW, Leidraad Zee- en Meerdijken, Delft 1999
TAW, Van overschrijdingskans naar overstromingskans, Adviesrapport en achtergrondrapport, juni 2000
Tielrooij, F. e.a., Waterbeleid voor de 21e eeuw, Advies van de commissie waterbeheer 21e eeuw, 2000
Ven, van de, G.P. et al., *Niets is bestendig, de geschiedenis van rivierovertoppingen in Nederland*, Utrecht, 1995


Vrijling, J.K., *Uncertainties emergency storage areas*, 2002

Vrisou van Eck, N. en M. Kok, *Standaardmethode Schade en Slachtoffers als gevolg van overstromingen*, Delft, 2000


*Wet op de waterkering*, 1995

EXPLANATORY WORD LIST

**Breach**
A gap in a dike which originates when the dike fails and widens due to the eroding water flow through the breach.

**Design discharge**
River discharge with a return period of 1250 years.

**Dike ring**
The closed system of water defences around a dike ring area.

**Dike ring area**
An area that must be protected by a closed system of primary water defences (dikes, dunes, special structures) and/or high grounds against flooding.

**Dike section**
Along one dike section the strength and load values can be assumed constant in space.

**SOBEK**
Computer program, which calculates stationary, and non-stationary open channel flow.

**Emergency storage area**
At times of high river discharges water is stored in special areas (emergency storage areas) to lower the river water level downstream.

**Failure mechanisms**
The way in which a dike can collapse (with subsequent development of a breach).

**High grounds**
Parts of the land, which are high and wide enough to retain the water.

**Inundation**
Flooding of an area.

**NAP**
Normal Amsterdam Level, about mean sea level.

**PICASO**
Pilot Case Overstromingsrisico’s. In this research project a detailed flood risk calculation is performed of dike ring area 43 (Betuwe en Tieler- en Culemborgerwaarden).

**Point of failure**
Predefined location in a river dike where failure of the dike can occur.

**Risk**
In this project defined as probability of failure x consequences.

**Storage volume**
The volume of water, which a dike ring area can contain in case of an inundation.