Stellingen bij het proefschrift

Naar een Operationele Aanpak van Overstromingsrisiko

van Hao-Ming ZHOU

september 1995

1. Het ontwerpen van maatregelen die de kans op overstroming beperken wordt gekenmerkt door verschillende doelstellingen en onzekerheden. De ondersteuning van dit ontwerpproces vereist een dynamische benadering van de veranderingen in het systeem in de tijd en een fasering van de implementatie.

2. De toepassing van geografische informatie systemen (GIS) bij de schatting van overstromingsschade kan de efficientie van het in stelling 1 bedoelde ontwerpproces sterk verhogen.

3. Een belangrijke bron van onzekerheid in de schatting van de waterhoogte en de herhalingsfrequentie in de zogenoemde 'overgangszone' in een rivier is het gebrek aan kennis over de gezamelijke kans in de tijd op het zich voordoen van een piek in het debiet en een piek in het zeewater niveau.


5. Bij financiële besluitvorming onder onzekerheid, kan onderschatting van de risico's leiden tot een financiële ramp, terwijl overschatting van de risico's kan resulteren in een verlies aan goede mogelijkheden.
6. De overeenkomst tussen risico management inzake overstromingen en risico management inzake aardbevingen is dat beide gefronteerd worden met grote onzekerheden. Deze onzekerheden zijn fundamenteel gerelateerd aan de problemen met voorspelling van het tijdstip, de plaats, de omvang, de herhalingsfrequentie en de effecten van de gebeurtenis.

7. Minstens drie belangrijke factoren hebben sterke invloed op de persoonlijke beslissingen inzake risico beperking: de ervaringen met risico's in het verleden, de financiële mogelijkheden en de persoonlijkheid.

8. Mogelijke effecten van klimaat verandering zijn niet enkel van invloed op het lange termijn gemiddeld gedrag van factoren in de hydrologische cyclus, maar mogelijk ook op de herhalingsfrequentie en de omvang van de extremen.

9. Een computer toepassing voor oplossing van een vraagstuk heeft zeer beperkte betekenis zonder begrip van het totale context van het vraagstuk. Er is een groot verschil tussen het specificeren van een reeks van stappen op basis van eenduidige regels en het identificeren van de context waarin de ingenieur denkt, rekt, interpreteert en ontwerpt. (gebaseerd op C.M. Savage: 5th Generation Management, Digital Press, USA, 1990, p. 194)

Propositions with the Ph.D. dissertation

Towards an Operational Risk Assessment in Flood Alleviation
- theory, operationalization and application

by Hao-Ming ZHOU

September, 1995

1. Flood alleviation planning and design are characterized by multiple objectives and uncertainty, and requires a dynamic approach to deal with changes of the system over time and the phasing of implementation.

2. Application of Geographic Information Systems (GIS) for flood damage assessment can greatly improve the efficiency of flood alleviation planning and design.

3. A major source of uncertainty in the estimation of flood magnitude and the occurrence probability in the so-called river transition zone is due to the lack of information on the joint probability of the timing of peak discharge and peak sea level.

4. Insurance programs do not create benefits in terms of reducing the expected consequence value of adverse events. Differentiated insurance charges are essential to ensure the trade-offs between benefits and risks.

5. In financial decision making under uncertainty, underestimating risks may lead to a potential financial disaster, while overestimating risks (conservative evaluation) may result in the lost of good opportunities.
6. With respect to risk management, one of the similarities between flood alleviation and earthquake reduction is the presence of various sources of uncertainty. These uncertainties are fundamentally related to the difficulties in the prediction of time, location, magnitude, occurrence frequency, and consequence of the adverse events.

7. At least three major factors influence the individual decision on hazard mitigation: the experience of the individual with hazard, the material wealth of the individual, and personality.

8. Possible consequences of climate change might not only affect the long term mean behaviour of elements of the hydrologic cycle but also both the frequency and the magnitude of extremes.

9. A computer application is extremely limited without understanding the "whole picture" of an engineering problem. It is one thing to identify a specific sequence of steps captured by rules, but is it quite something else to identify the whole context in which an engineer thinks, calculates, interprets and designs.


10. A father is the first and often the longest connection a daughter will have with a man. The father-daughter bond (or lack of bond) shapes her future relationships with male friends and lovers and influences how she moves out in the world.

TOWARDS AN OPERATIONAL RISK ASSESSMENT IN FLOOD ALLEVIATION

- theory, operationalization and application
TOWARDS AN OPERATIONAL RISK ASSESSMENT
IN FLOOD ALLEVIATION

- theory, operationalization and application

PROEFSCHRIFT

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in het openbaar te verdedigen ten overstaan van een commissie,
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Part I

Introduction and Background
CHAPTER 1

INTRODUCTION

1.1. Risk and safety

Human existence involves exposure to a wide variety of risks. These risks range from common activities such as driving cars to occupational activities, such as working in construction sites, and from natural disasters, such as earthquakes and floods, to man-made risks. Covello and Mumpower (1985) described nine kinds of changes, between past and present, in the nature of risks and in the means and ability of dealing with risks. The nature of risks has shifted from environmental hazards and infectious/epidemic diseases to new man-made risks, such as exposure to pesticides and radio active waste, which derive from advances in science and technology. Science and technology not only contribute to the mitigation, reduction and elimination of existing risks but also create new risks with a high catastrophe potential.

In recent times most of natural disasters are characterized by more damages but fewer deaths (White and Haas, 1975; Burton, et al., 1993). The increasing damage is associated with population growth and concentration, fixed capital accumulation and increased prosperity, as well as people's changing expectations and valuations. An earthquake or flood can result in much more economic damages than in the past, simply because of the accumulation and spatial concentration of property in disaster prone areas. Per capita losses however increased at a lesser rate and may have remained fairly constant (White and Haas, 1975). The decrease in catastrophic death is probably due to improvements in prevention, warning and evacuation.

Being aware of the earth's potential for disaster and witnessing increasing damages of natural disasters, the United Nations General Assembly adopted on December 22, 1989 the resolution 44/236, declaring the nineties as the "International Decade for Natural Disaster Reduction" (IDNDR). This UN resolution emphasizes a high degree of understanding of the impact of natural disasters and the need for prevention. In the UN resolution 46/182, dated December 19, 1991 and titled "Strengthening of the coordination of humanitarian emergency assistance of the United Nations", the essential importance of economic growth and sustainable development for the prevention of
natural disasters and other catastrophes was also emphasized (e.g. German IDNDR-committee, 1994).

The word risk, according to Covello and Mumpower (1985), originates from the Greek word "rhiza", which refers to the hazards of sailing around a cliff. Contemporary ways of thinking about and dealing with risks are different in many respects from earlier times. Major changes have occurred in the nature of risks that society faces, in the capabilities for risk analysis and assessment, and in the social and political context of risk management. Modern risk analysis has its dual roots in mathematical theories of probability and scientific methods for identifying causal links between different types of hazards and adverse effects (Covello and Mumpower, 1985).

*Risk* can be defined as the product of the probability of events and the magnitude of specific consequences (e.g. Lowrance, 1976 and Kasperson, et al., 1988). In this study, a definition of risk is employed, which is accepted by many authors (e.g. Keeney, 1980; Correia, 1987; Afshar and Mariño, 1990; Chin and Chittaluru, 1994): the probability of the realization of an adverse event. This definition is also adopted by the recently published Dictionary of Scientific and Technological Terms (Parker, 1994).

*Hazard*, a concept closely related to risk, is usually defined as something like a source of injury or damage (Kaplan, 1991). Therefore the difference between risk and hazard is that risks are the quantitative measures of hazards (Hohenemster et al., 1983). Thus aircraft usage is a hazard and the risk of dying in an aircraft accident is expressed in fatalities per passenger mile or trip.

*Safety* has been defined as the level of risk that is deemed acceptable (e.g. Lowrance, 1976; Kaplan and Garrick, 1981; Kaplan, 1991). In contrast with simplistic dictionary definitions this definition stresses that "safety" cannot be equated with "freedom from risk" or elimination of failure and damage. "How safe is safe enough" remains the fundamental question.

Risks may be divided into three tiers (Soby, et al., 1993). In the lower band, risks are readily accepted by the public because the benefits are felt to outweigh the disadvantages (Keeney, 1980). In the upper band, risks are regarded as completely unacceptable and must be reduced even at very high cost or, if not possible, the activities must cease. The intermediate region is one in which decisions on risk reduction are made by trading off associated costs and benefits.
Introduction

Risks are usually regulated by professional standards and/or legal regulations, which are commonly based on implicit trade-offs among costs and benefits of risk reduction. For example, design standards for flood defence projects protect people from flooding, standards for pollutant emissions protect the environment, and standards for equipment with radiation protect people from overexposure. Modification of safety levels of natural risks would be required when experience is accumulated over the long run on the consequences of extreme events. Catastrophes however have been a major source of change (White and Haas, 1975).

The growing damage of adverse events, the people’s increasing expectations of science and technology to reduce, mitigate and alleviate risks, and the increasing concern for safety, health and environmental protection imply a higher demand for safety and risk reduction. Revision of standards and regulations and improvements in planning and design practices are required to meet this challenge. Another important reason for the revision of standards is the acquirement of new knowledge and information. As the existing standard proves to be insufficient or the foundation of the standard is found to be erroneous, adjustment of the existing standards becomes inevitable.

The natural process system—the array of wind, water and earth processes—functions largely independently of human activities and is an object of scientific inquiry in its own right by for example meteorologists, hydrologists and geologists (Burton, et al., 1993). The interaction between natural processes and human systems may create hazards such as earthquakes and floods. All risks except from some planetary and astronomical events (e.g. meteorite strikes) are avoidable or at least can be minimized at some costs (Rowe, 1977). Natural disasters such as earthquakes, tornados and floods can be avoided by settling in areas that have lower or no exposure to these disasters. Man-made risks are avoidable by direct control and avoidance of exposure. Flooding therefore cannot be considered a purely natural risk, since human vulnerability seldom results from purely natural states or events but largely depends on location decisions as well as human intervention. Flood protection measures, by encouraging settlement in flood-prone areas, may increase the catastrophe potential.

Hazards can be responded through either adaptation in the organization and processes of social systems, or in specific and conscious adjustments intended to reduce the costs or increase the net benefits of the hazard. Adaptation means long-term arrangement of activity to take account of the threat of natural extremes, while adjustment means all those intentional responses to cope with the risk and uncertainty of natural events. The
three major classes of response, as described by White and Haas (1975), are: modifying the cause of the hazard, modifying vulnerability to the natural events, and distributing the losses. The social system sets limits within which different adjustments/adaptations are selected. The interaction between the natural and human system is presented in Fig. 1.1.

Fig. 1.1. Hazards, adjustment and effects (adapted from White and Haas, 1975)

The willingness to accept the risk of major disasters appears to be moving to less tolerance for loss of life but differs remarkably between hazards. Some risks are less tolerable for the public and consequently much attention is paid in reducing their impacts. For example societies are more alert and committed to the reduction of loss of life from certain hazards, such as floods and tornadoes than other natural hazards with high catastrophe potential such as earthquakes, hurricanes, tsunamis and volcano eruptions (White and Haas, 1975).

Approximately 90 percent of the world’s natural disasters causing more than 100 deaths originate in four hazard types: flood (40%), tropical cyclone (20%), earthquake (15%) and drought (15%) (Burton, et al., 1993). Flooding, as one of the main natural disasters in many countries, constitutes a classic problem in risk assessment and management. Almost all of the great ancient civilizations such as China and Egypt directly intervened to mitigate the effects of natural disasters such as flood. Historical records indicate that, throughout history, governments have played a major role in developing and financing elaborate systems of flood control (Covello and Mumpower, 1985).

In making flood alleviation policy, the benefits and costs of risk reduction should be balanced (e.g. Soby, et al., 1993). The magnitude, the occurrence probability and the damage of flooding can be reduced by "structural" measures such as building dikes and
regulating river flow and "non-structural" measures such as flood predicting and warning and evacuation. A public project with a safety level which is either too low (under-design) or too high (over-design) is irresponsible from the perspective of the well-being of people living in the floodplain or involves an inefficient and inequitable allocation of resources.

This implies a challenge for many disciplines in terms of developing integrated frameworks or concepts for problem analysis, methodologies for supporting planning and engineering, strategies for implementing projects and plans and information systems for monitoring and maintenance.

This study intends to contribute in some aspects to this challenge by developing theory, operationalizing methodologies for problem analysis, and providing engineering solutions for flood alleviation focusing on river dike improvement.

The remainder of the present chapter will introduce risk assessment (Section 1.2); sketch flood problems and flood alleviation (Section 1.3); formulate the causative components of flood damages (Section 1.4); summarize the challenges in flood alleviation (Section 1.5) and formulate the objective and scope of this research (Section 1.6).

1.2. Risk Assessment

There are various, sometimes overlapping, definitions of risk assessment and management. For example, Haines and Leach (1984) defined risk assessment as the overall process of risk identification, quantification, evaluation, acceptance, aversion and management, while Parker (1994) interpreted risk management as the overall systematic approach to analyzing risk and implementing risk control. To avoid ambiguities of terms, the following definitions are used, as a useful means for communication rather than as universally accepted definitions.

Risk assessment is the scientific process of quantifying risk, while risk management is the managerial response based on the resolution of various policy issues such as acceptable risk. Risk management decisions are made by considering risk assessments within the context of political, social and economic realities. Such decisions are frequently controversial due to the difficulty in determining risks that are acceptable to the public (Chin and Chittaluru, 1994).
Different steps in risk assessment have been identified in the literature (e.g. Otway, 1977; National Research Council, 1983; Ganoulis et al., 1991-a; Shabman, 1991; Chin and Chittaluru, 1994). Risk assessment includes risk determination and risk evaluation, and risk management includes risk assessment and risk control as illustrated in Fig. 1.2 (Rowe, 1977).

Fig. 1.2. Steps in risk management (adapted from Rowe, 1977)

Risk determination involves the related processes of risk identification and risk estimation.

Risk identification is the process of observation and recognition of new risk parameters or new relationships among existing risk parameters, or perception of a change in the magnitudes of existing risk parameters.

Risk, at the general level, involves two major elements: the occurrence probability of an adverse event and the consequences of the event. Risk estimation, consequently, is an estimation process, starting from the occurrence probability and ending at the consequence values, as illustrated in Fig. 1.3.
Fig. 1.3. The process of risk estimation (adapted from Rowe, 1977)

Risk evaluation is a complex process of developing acceptable levels of risk to individuals, groups, or society as a whole. It involves the related processes of risk acceptance and risk aversion.

Risk acceptance implies that a risk taker is willing to accept some risks to obtain a gain or benefit, if the risk cannot possibly be avoided or controlled. The acceptance level is a reference level against which a risk is determined and then compared. If the determined risk level is below the acceptance level, the risk is deemed acceptable. If it is deemed unacceptable and avoidable, steps may be taken to control the risk or the activity should be ceased. The perception and acceptance of risks varies with the nature of the risk and depends upon many underlying factors. Lowrance (1976) identified a wide array of distinguishing characteristics which determine the perception and acceptance of different risks: voluntary or involuntary exposure, availability of previous experience, degree of knowledge (known or uncertain), (ir-)reversibility of consequences, capabilities for risk reduction. The risk may involve a "dread" hazard or common hazard, be encountered occupationally or non-occupationally, have immediate or delayed effects and may affect average or especially sensitive people.

Risk aversion is the control action, taken to avoid or eliminate the risk, regulate or modify the activities to reduce the magnitude and/or frequency of adverse effects, reduce
the vulnerability of exposed persons and property, develop and implement post-event mitigation and recovery procedures, and institute loss-reimbursement and loss-distribution schemes (Covello and Mumpower, 1985).

1.3. Flooding and flood alleviation

Flood hazards range from inland (riverine) floods, the primary focus of this study, to flooding in ocean shores, sea coasts and estuaries. A riverine flood is defined as any significant rise in the river level resulting from rainfall, snowmelt or other reasons (e.g. Acrcman and Farquharson, 1992). An inundation happens, mainly as the consequence of flooding, when water flows where it ought not to flow (Gumbel, 1958). There is no significant difference between the term flood and inundation except the degree of speed of the water entering an area. Extreme floods normally result in inundation of flood plains.

Floods occur mostly in the river forelands along a river channel which have been used for long time by the river to periodically discharge its flood waters, and are really part of the stream channel. This area is called the floodplain of the river, and its width is an indication of the magnitude of the flood discharges which have occurred in the past (Pickels, 1925). Historical data however underestimate the probability of extreme events, since these may not yet have had time to occur (White and Haas, 1975).

Floods and inundations have been threatening human life and property throughout history. Just during the past 100 years, more than 9 million people have perished from flooding (Avakyan and Polyushkin, 1991-a). The most disastrous flood in this century happened in China in 1938 and caused human life losses of half a million (German IDNDR-committee, 1994). Of late, total damages from floods and inundations have had a pronounced tendency towards an increase. At the start of the century, the average annual loss in the USA amounted to about US$ 100 million; in the 1980's, it already exceeded US$ 1 billion (Avakyan and Polyushkin, 1991-b). In the last ten years, floods resulted in at least an estimated economic damage of US$ 55 billion, and 15000 human life loss all over the world (German IDNDR-committee, 1994). The rapid increase of the flood damages were partly due to poor and subjective estimation of flood damage. For example, a published statement that an earthquake or a coastal hurricane "has caused property losses of X dollars" may differ from the true figure by a factor of two or three (Rowe, 1977; Horlick-Jone and Peters, 1991). Burton, et al. (1993) stated that the
**Introduction**

popular estimates show a bias towards overestimating losses in industrial countries and underestimating losses in developing countries or in an area remote from centres of government and mass media.

The continuous exposure of the population to flooding creates serious challenges to search for measures to limit flood damages. The necessity of flood alleviation is due to man’s attempt to utilize flood plains or other flood prone areas for more beneficial purposes. The floodplain however cannot be appropriated without paying a price, either in the form of flood control works or in the form of flood damages.

Flood control works are warranted only if the damage reduction brought by them is greater than their cost. The necessity of making a flood alleviation project sustainable, which means that none of the objectives of flood alleviation, environmental protection, and socioeconomic development should be ignored (Zhou and Van der Heijden, 1993-a), motivates the development of sound methodologies for flood alleviation in order to achieve a balance between these objectives. A risk assessment approach is adopted in this study for the decision analysis for flood alleviation.

To assess flood risks, the following four questions must be addressed: (1) What are the flood risk and flood damage, (2) What are available options, (3) what are the cost and adverse impacts of the options, and (4) what is the associated trade-off in terms of all damages, costs, adverse impacts and risks.

Risk assessment in flood alleviation can be presented as in Fig. 1.4.

Risk assessment and management support the development and implementation of flood alleviation policy. In flood alleviation multiple objectives are pursued:

- increase national economic efficiency and promote regional economic development by reducing vulnerability of flood prone areas and enabling beneficial use;
- protect human health and prevent premature death;
- avoid social disruption;
- maintain environmental quality and protect the natural landscape;
- conserve soil and water resources; and
- create an equitable distribution of risks and the costs and benefits of risk reduction.
Fig. 1.4. Risk Assessment in flood alleviation

In the course of risk assessment in flood alleviation, the analyst should provide a menu of choices with a risk-based damage-cost evaluation. The opportunity cost of risk aversion should be considered.

This approach can offer a consistent and conceptually complete design framework for planning and design in flood alleviation. This fits well into our age of increasing concerns of the socioeconomic and environmental issues and sophisticated capabilities in data gathering and processing. The practical application of the risk assessment approach in flood alleviation in this study focuses on river dike heightening and decision analysis in design development.

The study of flood alleviation, as studies of other types of risk, should start from the identification of causative components of floods and flood damages, and capture the entire chain from causal events, outcomes, exposures, consequences and consequence values. The next section focuses on these causative components.
Introduction

1.4. Causative components of floods and flood damages

Floods can be caused by natural phenomena such as tsunamis, earthquakes, or human disruption such as bombing a dam. However, mostly floods result from the interaction between uncontrollable natural conditions such as climate, weather and sea level, and the characteristics of a river basin such as the topographic, soil and underground water conditions of the river basin, land use patterns, and river morphology. Floods can also occur as the consequence of a failure of flood control works such as overflowing, instability and foundation failure of dams and dikes.

Flood damages are the result of the physical contact with fast flowing water (i.e. a building is devastated), or the mere presence of the flood/inundation water (i.e. goods delivery is hampered due to the flood disruption of transport systems). The composition of flood damages is illustrated in Fig. 1.5. The level of flood damages depends not only on the characteristics of a flood but also on the characteristics of the properties and infrastructure systems in the floodplain. A severe flood causes more damages than a less severe one in the same floodplain, while floods of the same severity would cause more damages in an urban area than in a rural area.

Fig. 1.5. Composition of flood damages

The factors which could influence (increase or decrease) either the flood frequency or the potential flood damage in an area are called, in this study, the causative components of flood damage. Flood conditions and flood alleviation measures are two important sets of intervening variables in causing flood damage.
1.4.1. Flooding conditions

Floods and other hazards can be classified according to causal agents (Mitchell, 1974), impacts (Foster, 1976), social aspects (Kreps, 1989; Drabek, 1989). Floods in different geographic areas such as in ocean shores, sea coasts, estuaries and inland, have different characteristics and require different alleviation measures. This study focuses only on inland floods. For inland floods, analysis can be conducted on a natural unit such as a river basin, a river and its tributaries, a flood plain, a polder or system of polders (as is done in this study), and a dike-ring and its segments; as well as on administrative units such as a factory (as is conducted in De Boer and Zhou, 1994).

If a river basin has the sea as its downstream boundary, it can be divided into three hydrologic zones. The area where the hydrologic variables are only determined by the river discharge is called a river regime. The area where the hydrologic behaviours are dominated only by the sea tidal movement is called a tidal zone. The area in between, where boundary conditions are determined by both the river discharge and the sea level, is called as a river transition zone. Flood conditions vary with the zones of a river basin. In the tidal zone, flood threats come usually from a storm surge, which is mainly induced by strong winds. In the river regime, an exceptional weather condition could cause heavy rainfall and consequently high runoff and high water levels. In the transition zone a combination of high sea level and/or flood discharge could cause high water levels. The hydrological characteristics of the three zones can be summarized as in Table 1.1.

Table 1.1. Tidal, river transition and river zone

<table>
<thead>
<tr>
<th>hydrologic regime</th>
<th>dominating hydrologic boundary condition</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>sea level</td>
</tr>
<tr>
<td>tidal zone</td>
<td>×</td>
</tr>
<tr>
<td>river transition zone</td>
<td>×</td>
</tr>
<tr>
<td>river regime</td>
<td>×</td>
</tr>
</tbody>
</table>

The entire chain of causative events-outcomes-exposures-and consequences for inland flooding is summarized in Fig. 1.6. This causal chain can help in identifying flood conditions and formulating flood alleviation measures.
Fig. 1.6. The causal chain of flood damages

non-structural measures:
- flood prediction
- flood warning
- evacuation
- rescue

rainfall, snowmelt
river basin characteristics
reservoir
river morphology
channel improvement
channel diversion

river discharge
wind

wind, sea level

high water in transition zone
high water in river regime

flood defence
flood defence
flood defence

failure of defence works
failure of defence works
failure of defence works

tidal zone characteristics
transition zone characteristics
river regime characteristics

flood in tidal zone
flood in transition zone
flood in river regime

flood exposure
flood damages
Chapter 1

In principle, uncontrollable natural conditions such as wind, sea level and rainfall or snowmelt cannot be directly manipulated but can, to some extent, be predicted.

An extraordinary uncontrollable natural condition, such as a storm surge, heavy rainfall and/or snowmelt, results in high levels of partly controllable natural conditions, such as river discharges and water levels. Natural conditions can interact with each other: a heavy rainfall produces large discharges and high water levels in rivers, winds create higher waves with the increase of water depth.

The characteristics of river basins and river morphology can be modified in such a way that floods and their occurrence frequencies are reduced: forestation and soil and water conservation can increase the retention capacity of a river basin and thus decrease flood peaks. Channelization, channel improvements and channel diversions improve discharge drainage. The levels of hydrologic variables such as river discharges and peak water levels can, to some extent, be controlled: reservoirs reduce peak flows, river dikes and flood protection walls confine the flow within channels; retention basins and the floodway help to detent and divert excess flow.

It is generally expected that, in the future, the climate will change due to deforestation, land use changes and anthropogenic emissions of so-called greenhouse gases, of which CO₂ plays a dominating role (e.g. Pohlmann and Sündermann, 1994). The sea level rise is considered one of the major consequences of global warming. Long term climate change would drastically effect other variables such as rainfall as well.

The sea level rise can significantly affect the stage-discharge relationship of a river, since the sea level rise would result in a higher water level at a given level of flood discharge value (Stakhiv, et al., 1991). In addition to the sea level, coastal erosion is another natural process which increases flood frequencies. Coastal erosion directly decreases the capability of sea defence works and indirectly decreases that of river flood control works through the penetration of sea waters to inland through various estuaries, interfering with river behaviour. The sea level rise can increase some coastal erosion processes as well (e.g. Louisse and Verhagen, 1990).

The outcome of causative events and ultimate consequence of flooding can be intervened by a combination of protection works ("structural" measures), and organizational, financial and regulatory measures ("non-structural" measures).
1.4.2. Flood alleviation measures

Different kinds of flood conditions can be intervened with different kinds of flood alleviation measures as summarized in Fig. 1.7.

Fig. 1.7. A summary of flood alleviation measures

Preventive as well as emergency and corrective measures can be employed in flood alleviation at different stages of flooding, respectively, before, during and after flooding (Table 1.2).

**Structural measures (protection works)**

Different types of structural measures are applied in different zones of a river basin. Structural measures in the tidal zone such as dunes, sea dikes and storm surge barriers confine the sea; structural measures in the river regime and in the transition zone, such as reservoirs, dikes (levees), flood protection walls, retention basins, channel improvements, or channel diversion, confine or alter the flow stream of rivers and
channels. Structural measures can also change the spatial and time distribution of hydrologic variables. For example a reservoir can change flows by reducing peak discharges, while a channel diversion can alter destinations of flood water.

Table 1.2. Flood alleviation measures versus flooding stages (Pols, 1995)

<table>
<thead>
<tr>
<th></th>
<th>preventive measures (before flood)</th>
<th>emergency measures (during flood)</th>
<th>corrective measures (after flood)</th>
</tr>
</thead>
<tbody>
<tr>
<td>structural measures</td>
<td>dike, storm surge barriers</td>
<td>sand-bagging, emergency repair of dike</td>
<td>repair, restoration and reconstruction</td>
</tr>
<tr>
<td>non-structural measures</td>
<td>land use zoning, flood proofing</td>
<td>warning system, evacuation, rescue</td>
<td>insurance, relief and rehabilitation</td>
</tr>
</tbody>
</table>

When the levels of hydrologic variables exceed the safety level of a flood defence structure, a flood may occur. High sea levels, strong winds, large discharges, and high river water levels may further result in failure of flood defence structures and consequently floods. Various structural measures can fail due to different failure mechanisms and the functioning of some structural and nonstructural measures can be interfered by management error and human error.

Two features of structural measures have been reported in the literature. First, protection works do not provide complete protection against flooding (Thampapillai and Musgrave, 1985; Williams, 1994). They only directly reduce the occurrence frequency and indirectly reduce expected flood damages. Second, structural measures can often create a false sense of security. The belief that the occurrence frequency of flooding has been reduced could lead to an intensive development of the floodplain. Consequently, the potential flood damage may tend to increase (White and Haas, 1975; Thampapillai and Musgrave, 1985).

Since the 1970’s, the U.S. policy regarding floods shifted from a primary orientation on flood alleviation by protection works to a broader scope including non-structural measures (e.g. Changnon, 1985; James and Hall, 1986). According to Denning (1993), one of the lessons learned from the 1993 Mississippi River flood is the need to look beyond protection works in dealing with the floodplain management problems. The recent flood threats and flood damages in the river basin of the Rhine and the Meuse have stressed the need for integral basin management as well.
Introduction

Non-structural measures

Non-structural measures usually reduce the value of flood damages, and can be applied to all hydrologic zones. These organizational, financial and regulatory policy measures include: land use planning and zoning, flood proofing, flood prediction and warning systems, evacuation and rescue programs, and flood insurance or compensation programs.

Floodplain land use planning and zoning involve the imposition of development control on the floodplain in order to reduce the expected value of flood damage (Thampapillai and Musgrave, 1985; CCIW, 1988). Zoning of the floodplain based on vulnerability and the frequency and severity of flooding is required. Land use zoning in the floodplain may restrict new development, encourage the transformation of existing uses of the damage-prone areas, and induce relocation.

The value of flood damages can be reduced by flood proofing, which, as defined by Scheaffer (1960), is a body of adjustments to building content and structures. There are four sets of general approaches to flood proofing: raise or move buildings, construct barriers to stop flood water from entering buildings, modify building structures and relocate contents to minimize flood damage (Edward and Thomas, 1993), and select water resistant materials. Flood proofing regulations can be incorporated in zoning ordinances and building codes.

Flood predicting and flood warning have received increasing attention because of its effectiveness in reducing the value of flood damages by precautionary measures. White (1975), as referred by Melching (1991), noted that a warning system might yield a benefit-cost ratio of at least 5 in most cases and also provide many non-economic benefits. As only a portion of the floodplain property is left exposed to floods, the expected flood damage is reduced. Flood predicting and warning are particularly effective for the reduction of human life losses, as well as social disruption and property damage (White and Haas, 1975). In the course of flood, property damages and human life losses can still be mitigated by flood evacuation and rescue programs.

Flood insurance and compensation programs do not create benefits by reducing the expected value of flood damage and only distribute the losses (Thampapillai and Musgrave, 1985).
White and Haas (1975) indicated that some flood alleviation measures may interact with each other, through either a strong (close) or a weak (loose) linkage. For instance, if a community adopts land use regulations in a flood prone area, it is less likely to seek control and protection works for the same area. On the other hand, if the community already benefits from a levee or an upstream control work, land use zoning in the remaining vulnerable zones of the floodplain would be less likely to be supported. Structural and non-structural measures may have to be combined to effectively reduce flood damage, since, for example, warning is usually more effective in reducing human life loss than structural measures.

1.5. Challenges in flood alleviation

The above discussion reveals the following challenges in flood alleviation:

(1) Risk assessment and risk management should cover the entire causal chain of causative effects, outcomes, exposure, consequences, and consequence values and perception.

(2) A decision framework in flood alleviation should be developed, incorporating both structural and non-structural measures. The framework should be capable of coping with different kinds of flood damages and the costs and adverse impacts of alleviation measures. The implementation timing of the measures should also be considered.

(3) A complete trade-off between the flood damage and the cost and adverse impacts of flood alleviation is required. Most flood alleviation studies are based on economic flood damages and the investment cost of upgrading flood protection. Social and environmental issues and intangible flood damages are not yet readily incorporated into the trade-off between the damage, cost and adverse impact.

(4) Interdependence of flood alleviation measures is to be incorporated. Structural measures intervene the river hydraulic conditions, i.e. there is a hydrologic interdependence of flood alleviation measures such as dike heightening. Consideration of this interdependence in decision analysis is required. The river transition zone is an area where such a hydraulic interdependence is more obvious and difficult to deal with due to the interaction of the up- and downstream boundary conditions.

1.6. Objective and Scope of this study

The overall objective of this study is to support the decision making in flood alleviation,
Introduction

through the formulation and the operationalization of a risk assessment approach, focusing on river dike heightening, with an application to a system of polders in the transition zone in the Netherlands.

The specific objectives of this study are as follows:

- To review the flood history and the studies for flood protection in the Netherlands, and to identify issues for further study (Chapter 2);

- To present design tools which are currently used in flood alleviation such as reliability based design, economic optimization and multiobjective evaluation (Chapter 3);

- To develop the framework of a dynamic multiobjective decision analysis model under uncertainty for flood alleviation focusing on dike heightening, and to provide a formulation and practical solution of the optimization problems involved (Chapter 4);

- To develop methods and procedures for the estimation of the flood magnitude and its occurrence probability under hydrologic boundary conditions of unsteady states in the transition zone (Chapter 5);

- To review and develop assessment methods for various kinds of flood damages, such as direct and indirect, and tangible and intangible effects. To develop a Geographic Information System (GIS) assisted approach for a detailed inventory of flood damage to property, and thus to assist the assessment of tangible flood damages with a large spatial coverage (Chapter 6);

- To identify various uncertainties in flood alleviation and characterize methods for overall uncertainty analysis (Chapter 7);

- To apply methods operationalized in the previous chapters to four neighbouring polders, which covers a total surface area of more than 500 km², in the transition zone in the Netherlands (Chapter 8); and

- Finally, to summarize observations and conclusions of this study resulting in a view on further developments (Chapter 9).
CHAPTER 2

FLOODING AND FLOOD PROTECTION
IN THE NETHERLANDS

2.1. Introduction

The Netherlands, as mainly situated in the delta of the Rhine and the Meuse (Fig. 2.1) has to be protected not only from the storm surge from the North Sea, but also from the large discharge from the upstream river basins.

The river Rhine rises as a small glacial brook on Mount Sankt Gotthardt in the Swiss Alps. Gathering the water of many tributaries, it drains through France, Germany and the Netherlands into the North Sea. The Dutch part of the Rhine starts from the Dutch-German border near Lobith and ends at the North Sea at Hoek van Holland.

The river Meuse is a river which has its source in France and flows through Belgium to the Hollands Diep in the Netherlands. The Hollands Diep and the adjoining Haringvliet are artificial lakes separated from the sea by means of a dam. The Hollands Diep is generally chosen as the mouth of the Meuse (Berger, 1992).

The Rhine is a snowmelt and rainfall river basin, while Meuse is a typical rainfall basin. Some other important characteristics of the two rivers are listed in Table 2.1.

In this chapter, the Dutch history of flooding and flood protection system will be first introduced (Section 2.2 & Section 2.3). The studies of flood protection in the Netherlands will be reviewed to present the state-of-the-art in practical applications (Section 2.4) and clarify the issues for further study (Section 2.5).

2.2. History of flooding

There were at least 140 floods with casualties between AD 1200 and 1953 in the Netherlands (De Haan, 1990). One of the most serious floods happened in February, 1953. A storm flood in the coastal area devastated an area of about 1300 km² in the southwestern part of the Netherlands, and resulted in more than 1800 people losing their
lives, with more than 72,000 having to be evacuated. The total estimated figure of the material damage and loss was at least 1.1 billion Dutch Guilders (Delta Committee, 1962).

Fig. 2.1. The Netherlands and the river basin Rhine and Meuse

Table 2.1. A comparison between the Rhine and the Meuse

<table>
<thead>
<tr>
<th>Characteristics</th>
<th>the Rhine</th>
<th></th>
<th>the Meuse</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Catchment</td>
<td>Dutch Part</td>
<td>Catchment</td>
<td>Dutch Part</td>
</tr>
<tr>
<td>Catchment area in km²</td>
<td>185,000</td>
<td>25,000</td>
<td>33,000</td>
<td>11,740</td>
</tr>
<tr>
<td>Length in km</td>
<td>&gt; 1000</td>
<td>167</td>
<td>874</td>
<td>±200</td>
</tr>
<tr>
<td>Max. known discharge (m³/s)</td>
<td>12280 at Lobith</td>
<td></td>
<td>3000 at Borgharen</td>
<td></td>
</tr>
<tr>
<td>Mean discharge (m³/s)</td>
<td>2200 at Lobith</td>
<td></td>
<td>230 at Borgharen</td>
<td></td>
</tr>
</tbody>
</table>

sources: Cazemier, 1988; Berger, 1992
Flooding and flood protection in the Netherlands

The parliament then decided that this kind of disaster was intolerable and should be prevented in the future. The disaster led the Minister of Transport, Public Works and Water Management immediately to install the Delta Committee to investigate the hydrotechnical problems in the disaster area as well as other parts of the Dutch tidal area. As the result of the Delta Committee, the Delta Plan was adopted by law and implemented. In the past decades many dikes and dunes have been strengthened, estuaries have been closed off from the sea by dams and storm surge barriers were built in the Eastern Scheldt and New Waterway.

In the end of 1993, a flood from the Meuse, the second largest river in the Netherlands, occurred in France, Belgium and the Netherlands. The flood and inundation in the Netherlands resulted in an estimated total material loss of about 250 million Dutch Guilders and about 8000 people were evacuated. The perception of the emotional damage was described as "very great" (MTPW, 1994).

In January 1995, a serious flood occurred in Germany, France, Belgium and the Netherlands. The flood which occurred in the Dutch part of the river basin of the Rhine and the Meuse was the most serious one since the flood in 1953. Several dikes broke and polders were flooded, dikes had to be inspected every hour due to water load and soaking of soil. Rescue teams were standing by to strengthen dikes with protective covering and sand-bags to maintain the hydraulic and geo-mechanical stability if possible dike deterioration/failure was identified. The estimated direct and indirect damages were 200 million and 1.0 to 1.5 billion Guilders, respectively, and 250,000 people and millions of livestock were evacuated. The inhabitants thought that they deserved to be better protected from floods. The prime minister called for another "Delta Plan" to improve the river dikes.

2.3. The Dutch flood defence system

About half of the Netherlands lies below the mean sea level and has to be protected by dikes and natural dunes from flooding by sea and rivers. When a part of delta areas, lagoons, flood plains, or any level area which was originally subject to a high water level, either seasonally or permanently and due to surface or ground water, is separated from the surrounding hydrological regime, it becomes a polder (Segeren, 1983). In this way its water level can be controlled independently of the surrounding regime.
Chapter 2

As far as is known, the construction of flood protection works started approximately in the third century before Christ when people in the northern part of the Netherlands built artificial mounds to live on. In the first century, under the influence of the technical capabilities of the Romans, people began to construct small dams along the rivers. Later dams were also constructed to connect several mounds. These dams could resist the regular floods and protect the lands, to some extent, against flooding, but they were certainly too low to safeguard the land under extreme conditions. Large activities in dike construction during the seventh and eighth century made it reasonable to expect that in the eighth century the first polders were constructed. During the following centuries the protection against flood was improved phase by phase (Schultz, 1983).

The Dutch flood defence system consists of dunes, sea dikes, storm surge barriers, sluices and river dikes. If one of the elements fails, the polder will be flooded and inundated. The main fault tree is presented in Fig. 2.2.

![Fault Tree Diagram](image)

Fig. 2.2. The main fault tree of the Dutch flood defence system

The low-lying area consists of 53 polders; the surrounding dikes protect the land areas they enclose from flooding. A polder is protected from flooding by enclosing dike sections and some hydraulic structures such as a sluice. There are currently approximately 3440 km of dikes, of which about 570 km pass through villages and towns. In total, 13 km of dike reach is characterized by cultural-historical buildings of high value at both sides (Peerbolte, et al., 1991). Dikes play a crucial role in flood protection in this country and have formed a unique dike landscape.

Since the flood in 1953, storm surge barriers were built in the Eastern Scheldt and New Waterway, dunes were improved, and dike improvements (heightening and strengthening) have been implemented. Construction of sea dikes and storm surge barriers requires substantial investments, but meets less societal resistance than river dike improvements. Since the 1970's, protest grew against the harmful effects on the
Flooding and flood protection in the Netherlands

landscape and natural and cultural assets on and along river dikes. It was felt that the
dike improvement measures were too strongly dominated by limited technical
engineering approaches based on traditional design criteria lacking attention for the
social and environmental impacts and alternatives (Zhou and Van der Heijden, 1993-b).
This protest represents the changing societal appreciations of social and environmental
assets.

2.4. An overview of flood protection studies in the Netherlands

This study builds upon the accumulated results of previous research on flood alleviation
in the Netherlands. A wide range of studies has been carried out on flood protection of
polders, and dike improvement in particular, through which a good understanding of
these problems has been reached. Most of the studies relevant to dike heightening
focused on the extrapolation of hydrologic variables at boundaries, the establishment of
the protection level of polders, the estimation of the probability of all failure
mechanisms of a polder, the assessment of flood damages, and analysis of the various
uncertainties in flood alleviation.

2.4.1. Extrapolation of the hydrologic boundary condition

For the estimation of potential flood damages, magnitudes of hydrologic variables at the
sites of interest along a river and their occurrence frequencies are required. The
probability curves at boundaries (gauging stations) are the basis to calculate the
probability curves at these sites.

Lorentz, the Dutch physicist and Nobel-price winner, initiated the application of
probability theory to fluctuations in the water level in the 1930's.

Wemelsfelder (1939) drew attention to the fact that the logarithm of the main number
of high tides per annum exceeding a given level, if plotted against that level, leads to
a curve which, in the middle of the range of observations, approaches a straight line, the
curve becoming more vaguer (less precise) towards the higher high tides. This was the
first extrapolation of the probability curve for high tide in the Netherlands.

Van Dantzig and Hemelrijk (1962) plotted all observations of sea levels in the same
way, and found that both the Exponential and Log-Normal Curve fitted observations
quite satisfactorily. The two curves hardly differed, even if one extrapolated them to 6 meters above NAP\(^1\). Due to its simplicity, the Exponential distribution was recommended. A Gumbel distribution was also applied to capture some points. Their early research led to accepting the "working line" of the probability curves. According to the working line, the sea level of 5 meter at Hoek van Holland, a hydrologic gauging station where the Rhine enters into the North Sea, is associated with an exceedance probability of \(10^{-4}\) per year.

In 1977 the River Dike Commission (also named the Becht Commission) estimated the probability curve of the river discharge at Lobith, a hydrologic gauging station of the Rhine near the Dutch-German border, with various probability distributions, namely, Linear Exponential, Normal, Log-Normal, Pearson III, Log-Pearson III and the Gumbel distribution. One of the findings was that the Exponential distribution can be applied to the series of discharges exceeding 5000 m\(^3\)/s. Based on this finding and the estimates with other probability distributions, a "working line" of the probability curve of discharge was produced, in which the exceedance frequencies were estimated for the discharges of 18000, 16500, 15200 m\(^3\)/s, respectively, 1/3000, 1/1250, 1/500 per year (MTPW, 1977).

In 1992 the Boertien Commission (also called Boertien I) reviewed the probabilities of the discharges at Lobith. In this study, the effects of the changes in the German part of the Rhine were incorporated. Several probability distributions such as Gumbel, Pearson III, Log-Normal and Pareto type (details on this type, see Gumbel, 1958) were used. Since none could be regarded to be superior, the recommended discharge quantile of a particular occurrence frequency was obtained, based on the average from several distributions. The exceedance frequencies of 1/1250, 1/500, 1/200 for, respectively, three discharge quantiles 15000, 14100 and 13000 m\(^3\)/s were recommended. The Pareto type (probability distribution) was implicitly suggested as well (MTPW, 1993).

Recently the probability curve at Hoek van Holland was reviewed using both parametric (specifying a probability distribution function) and non-parametric distribution approaches and different probability estimation techniques. The sea levels corresponding with the exceedance frequency of \(10^{-4}\) were estimated by different approaches and

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\(^1\): NAP (Nieuw Amsterdams Peil) is the Dutch reference level for altitude; all altitudes in this chapter and in Chapter 8 refer to NAP.
techniques. The conclusion was drawn that the sea level of 5 meter with an exceedance frequency of 1/10000 estimated by a non-parametric approach appeared to be the most "attractive" estimate (Dillingh, et al., 1993), and the Pareto type was selected as the most appropriate parametric distribution function (Philippart, et al., 1994).

The studies on the extrapolation of probability curves at Hoek van Holland and Lobith can be summarized in Table 2.2.

Table 2.2. A summary of practices in the extrapolation of boundary conditions

<table>
<thead>
<tr>
<th>study</th>
<th>Hoek van Holland (sea level in meter)</th>
<th>Lobith (river discharge in m³/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>methods</td>
<td>recommendation</td>
</tr>
<tr>
<td>Wemelsfelder</td>
<td>no, but consideration of extrapolation</td>
<td></td>
</tr>
<tr>
<td>(1939)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dantzig &amp; Hemelrijk</td>
<td>Exponential, Log-Normal, Gumbel</td>
<td>working line</td>
</tr>
<tr>
<td>(1962)</td>
<td></td>
<td>5</td>
</tr>
<tr>
<td>Becht Commission</td>
<td>Linear, Exponential, Normal, Log-Normal, Pearson III, Log-Pearson III, Gumbel</td>
<td>working line</td>
</tr>
<tr>
<td>(1977)</td>
<td></td>
<td>16500</td>
</tr>
<tr>
<td>Boertien I</td>
<td>Gumbel, Pearson III, Log-Normal, Pareto Type</td>
<td></td>
</tr>
<tr>
<td>(1993)</td>
<td></td>
<td>14100</td>
</tr>
<tr>
<td>Dillingh, et al.</td>
<td>parametric and non-parametric approach</td>
<td>5</td>
</tr>
<tr>
<td>Philippart, et al.</td>
<td></td>
<td>13000</td>
</tr>
<tr>
<td>(1994)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

2.4.2. Acceptable protection level by economic optimization

Efforts to establish design standards of a polder's dike height can be traced back, again, to the studies of the Delta Committee in the early 1960's. The maximum sea level at Hoek van Holland in the 1953 disaster was 3.85 meter. According to a meteorologic study, the storm could have produced a maximum sea level of 5 meter if the various
factors which can all determine the storm effect (intensity, direction and duration), would have coincided unfavourably. Therefore, given the importance of "the heart of the Netherlands" (the area formed by the lines between Amsterdam, Rotterdam and Utrecht, which is called Randstad in Dutch), the Delta Committee took the sea level of 5 meter at Hoek van Holland as the basic design standard (Delta Committee, 1962).

The Delta Committee also studied this basic design standard by means of economic calculation. Van Dantzig and Kriens (1960) investigated the economically optimal dike height under the following assumption: if and only if the water level exceeds the dike crest, all (capital and consumption) goods in a polder will be destroyed, i.e. the probability of the loss is equal to the probability of exceeding the dike crest. The total flood damages include the goods in the polder and all the sequential damage suffered when the goods in the polder are lost. Both the exceedance probability and the total loss were assumed to be constant over time. The flood damage and the cost of dike heightening were assumed to be the functions of the additional dike height (the extra height added to the existing dike). The function of net benefit versus the added dike height was therefore optimized.

In the case study for the polder of Central Holland, economic growth and the dike settlement were considered as well. The flood damage estimate was based on the inventory of capital goods and the increasing stock of the (lasting) consumption goods in the private sector.

The design level of 5 meter at Hoek van Holland was also considered economically justified (Delta Committee, 1962). The issues of human life losses, the intangibles and the property discounting were excluded in the studies of the Delta Committee.

The Committee recommended design standards, in terms of exceedance frequency, of 1/10000 (per year) for "the heart of the Netherlands", and 1/4000 (per year) for other areas of the Netherlands.

In 1975 the Becht Commission was installed to establish the flood protection standard for the river area (MTPW, 1977). Since, the river area is outside "the heart of the Netherlands", according to the Delta Committee, the safety level of 1/4000 should apply.

In the study of the Becht Commission, five hydrologic boundary conditions were considered, which are the combinations of three discharges at Lobith with their
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Exceedance frequencies of 1/500, 1/1250, 1/3000 (per year) and hydrologic conditions in other relevant river branches. Accordingly five dike improvement alternatives were formulated, each of which should heighten the existing dike up to a level corresponding with the water levels under the five hydrologic conditions. The economic flood damages were estimated, assuming that the area will be flooded up to the level of the 1926's flood (the corresponding discharge was 12500 m³/s at Lobith). The Commission had made a thorough inventory and valuation of a large number of the landscape, natural and cultural assets within a 50 meter zone alone both sides of the river dikes.

Recommendations were made on the basis of the estimated economic flood damages, the economic cost of each dike improvement alternative, and the landscape, natural and cultural impacts ("LNC") of dike improvement alternatives. One of the recommendations was that the dike design standard for the river area should be 1/1250, corresponding to a discharge of 16500 m³/s at Lobith. It was the first time that the LNC impacts of dike improvements were considered.

Appreciation of the environment increased since the 1970's. To reconcile the demands for safety and preservation of buildings and landscape, the Boertien Commission (MTPW, 1993) reviewed the design standard for river dikes along the non-tidal river branches. This study covered 8 polders with a total bottom surface area of more than a few thousands km².

The study of Boertien I included three alternative safety levels, namely, 1/200, 1/500 and 1/1250 per year. Innovative designs for dike improvement were proposed in this study. For the 1/500 and 1/1250 level four design strategies were considered, namely, "conventional", "improved", "selective smart" and "very smart" design strategies. A smart design is to provide safety from flooding and to reduce adverse environmental impacts due to dike improvement. In this study, the impacts of dike improvement on LNC were considered as a part of the dike improvement cost (Walker, et al., 1993 and 1994). It was assumed that there were no substantial changes in the valuation of LNC impacts since the Becht Commission, and thus the data and valuations of LNC impacts of the Becht Commission were reused.

Upgrading from 1/500 to 1/1250 was found to result in only a 5% increase in LNC impact. The potential flood damages of each alternative were estimated based on the following assumptions: the polder dike should be upgraded in correspondence with each protection level; in the event of a dike failure, water flows into a polder and starts filling
it up to the lowest design dike height. The thoughts behind the second assumption was that the river flow is sufficient to fully fill polders. Obviously, the volume of river flow available for the filling of polder depends on the boundary conditions, and whether polders would be fully filled depends also on the retention capacity of the polders and the soil penetration capability during a flood.

The commission considered it practically impossible to predict which polders would be flooded and which ones would remain dry when the river level exceeded the design dike height, neither would it be possible to assign probabilities to the various outcomes. Consequently, three flood scenarios were specified, in which only one, three and all eight polders, respectively, would be fully flooded. The consideration of different dike design strategies and the inclusion of the scenarios of flooding were two advances on the previous study by the Becht Commission.

In 1994 the Boertien Commission (also called Boertien II) studied, among others, the possible structural measures for flood alleviation mainly in the province of Limburg located in the south-east of the Netherlands, which suffered a serious flood at the end of 1993. The Meuse running through Limburg is a river without dikes. Floods occur in the floodplain.

Deepening and widening of the river and its foreland were considered, with three safety levels of 1/250, 1/500, 1/1250 (per year). Five protection "strategies" were formulated. In this study, the flood damage on materials, the landscape and ecologic impacts of the structural measures and their cost were compared with the do-nothing case. One of the improvements by Boertien II was that the hydrologic and hydraulic uncertainty and uncertainty in the estimation of the flood material damage and implementation cost were analyzed. The uncertainty in the net benefit of protection alternatives was quantified, as the overall combination of the uncertainties in each individual source. It was found that in this specific area the uncertainty in flood material damages was larger than that in costs. The uncertainty analysis and the consideration of structural measures other than dike improvement were two areas of significant progress in this study (MTPW, 1994).

2.4.3. Acceptable protection level for human life

For the design of structures which might bring risks to human life in case of failure, the so-called "scale" effect of how many people might be killed in case of failure plays an essential role in the determination of the acceptable level of risk to humans. If the
failure of a structure can result in more human life loss, the structure should be designed at a lower failure probability.

The following equation for the determination of the so-called rational risk level, presented by CIRIA (1977), can be regarded as an attempt to consider the social (scale) effect in designing structures.

\[ P_f = \frac{10^{-4} K_s n_d}{n_r} \]  \hspace{1cm} (2.1)

in which: \( P_f \) is the probability of failure due to any cause during the design life \( n_d \) years, \( K_s \) is a social criterion factor to distinguish structure types and \( n_r \) is the number of people at risk in the event of a failure.

The social criteria factors for some types of structure are given in CIRIA (1977) as shown in Table 2.3. Actually the risks of failure on human life are implicitly balanced by decreasing the factor with the increase of the structure risk to human life in case of failure of the structure.

Table 2.3. Social criterion factor

<table>
<thead>
<tr>
<th>nature of structure</th>
<th>( K_s )</th>
</tr>
</thead>
<tbody>
<tr>
<td>places of public assembly, dams</td>
<td>0.005</td>
</tr>
<tr>
<td>domestic, office or trade and industry</td>
<td>0.05</td>
</tr>
<tr>
<td>bridges</td>
<td>0.5</td>
</tr>
<tr>
<td>towers, masts, offshore structures</td>
<td>5.0</td>
</tr>
</tbody>
</table>

The concept of this rational risk level has been further divided into an individually and socially acceptable level, and each of them has been operationalized for flood alleviation (Vrijling, 1985).

The individually acceptable level is the risk that a member of the community on the average is prepared to accept. An individual determines the individually acceptable level of flood risk on the basis of his own experience or the reported experience of others. The degree of voluntariness with which the risk is endured plays a very important role in the personal acceptance of risk. Generally speaking, individuals are more tolerable to risks taken in a voluntary activity such as mountaineering, than those taken involuntarily.
Flooding should be considered as an involuntary event, since people usually do not have full freedom in choosing their living and working sites.

The socially acceptable level of risk is based on the assumed risk aversion model, which leads to an evaluation of the acceptable probability of failure depending on the number of independent places where the activity under consideration is carried out.

The method suggested by Vrijling (1985) to calculate the individually and socially acceptable level of risk, respectively, is as follows:

\[ P_s(i) < \beta \frac{10^{-4}}{P_{d\|i}} \]  
(2.2)

\[ \sqrt{P_s(i)} < \frac{100 \beta \sqrt{N_s}}{k \ P_{d\|i} \ N_p} \]  
(2.3)

in which: \( P_s(i) \) represents the individually acceptable level of risk related to activity "i" per year; \( \beta \) is the policy factor, varying with the degree of voluntariness with which the activity is undertaken, ranging from 10 in the case of complete freedom of choice to 0.1 in the case of an imposed risk; \( P_{d\|i} \) is the probability of being killed in the event of an accident or an activity; \( P_s(i) \) is the socially acceptable level of risk related to activity "i" per year; \( N_s \) is the number of places where the activity under consideration is carried out; \( N_p \) is the number of people at risk; \( k \) is the risk-aversion-parameter ranging from 2 to 3.

It was suggested that if the individually and societally acceptable level of risk differ from the economically optimal level, then the most rigorous level should be adopted (TAW, 1990; Vrijling, et al., 1993), which is of course very attractive. The question remains if the society can sustain such rigorous levels for all kinds of risk because of the significant cost for risk reduction, and if not, how various risks can be reduced in an alternative way. In addition, since flood protection structures vary in levels of engineering design, maintenance, operational control and other factors, mechanisms of structural failure are difficult to establish even with detailed post flood information (Baecher, et al., 1980). Interpretation of historic failures is difficult because the quality of data is mixed and the causes of floods are not homogeneous. Similarly, calculations of the potential human life losses are very difficult. Therefore the estimate of \( P_{d\|i} \) is problematic.
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In 1989 a study was conducted by Vrouwenvelder and Wubs to review the safety level of a polder called Alblasserwaard, on the basis of individually and societally acceptable as well as economically optimal levels of flood risk. The Alblasserwaard polder stretches from the tidal zone through the river transition zone to the river regime (including thus three hydrological regimes). The official design standard for this polder is 1/4000 per year. In the study, breaching developments at a few pre-selected dike sections of a polder in each hydrological regime were investigated. The maximum width and depth of breaching and the time duration from the initial breach to the breach of the maximum width at different dike sections were assumed. The discharge flowing from the river through the breaching section into the polder was calculated and the inundation depths in the polder were predicted consequently. Based on the predicted inundation depth in the polder, the flood damages were estimated. The costs of upgrading the present dike to the height level corresponding with 1/4000 were estimated.

It was the first time that the discharge and the total amount of water from the river through the breaching dike section into polder were calculated, and the first time that such a study was conducted for a polder located partly in the river transition zone. However, in this study, it was assumed that the influence of the breaching discharge on the water level in the river a certain distance away from the location of breaching could be ignored. This implies that the river discharge is large enough to keep the water level in the river constant. It should be noted that this assumption and a similar assumption for the Boerdtien I (polders could be fully filled with water from the river) do probably hold for the Dutch polders, where the breaching/overflowing discharge is insignificant in comparison with the river discharge. In addition, it is difficult to rationally assume the maximum breaching depth and width.

From the three calculated levels of safety in each of the three hydrological regimes, the most rigorous level was selected for each regime. Based on the assumption that the hydraulic behaviour of each regime is independent from the other, the total probability of a polder breaching was calculated as the probability that at least one of the three regimes will be flooded due to breaching. Actually the hydraulic behaviour in the transition zone is unlikely to be independent from that in the tidal zone and that in the river regime. One of the achievements in this study and in the follow-up study (Vrouwenvelder and Wubs, 1992-a) was the standardization of flood damages as a function of the flood depth for some kinds of properties.
The studies on the design standard for flood protection of polders are summarized in Table 2.4.

Table 2.4. A summary of practices with respect to protection level of polders

<table>
<thead>
<tr>
<th>study</th>
<th>flood damage consideration</th>
<th>cost &amp; impact consideration</th>
<th>hydraulic simulation</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>economic</td>
<td>human</td>
<td>1</td>
<td>cost</td>
</tr>
<tr>
<td>Delta Com.</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Becht Com.</td>
<td>x</td>
<td></td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>Boertien I</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Boertien II</td>
<td>x</td>
<td>x</td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>Vrouwenvelder</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1: social and environmental damage; 2: smart design

The updated recommendation for the design standards is outlined in Table 2.5.

Table 2.5. Updated design standards for flood protection

<table>
<thead>
<tr>
<th>study</th>
<th>hydraulic regime</th>
<th>standard</th>
<th>hydrologic quantile</th>
</tr>
</thead>
<tbody>
<tr>
<td>Delta Committee</td>
<td>tidal zone (Randstad)</td>
<td>1/10000</td>
<td>5 m at Hoek van Holland</td>
</tr>
<tr>
<td></td>
<td>tidal zone (other area)</td>
<td>1/4000</td>
<td></td>
</tr>
<tr>
<td></td>
<td>river transition zone</td>
<td>1/4000</td>
<td></td>
</tr>
<tr>
<td>Boertien I</td>
<td>river regime</td>
<td>1/1250</td>
<td>15000 m³/s at Lobith</td>
</tr>
</tbody>
</table>

2.4.4. Overall probability of dike failure

Dikes can fail under various failure mechanisms, therefore, in the context of dike improvement the total probability of all failure mechanisms of a polder has to be considered. Vrijling (1987) suggested three steps for the study of the failure probability of a system of water-retaining structures. First, determine all possible mechanisms of structural failure and all other possible causes (e.g. management and human error). Second, analyze the relationship between all possible failure mechanisms and the ultimate consequences of flood with a fault tree. Third, determine the total probability of failure by means of a probability calculation.
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Vrouwenvelder (1993) conducted a similar study based on the probability theory of a series system. A polder system consists of different failure mechanisms, such as overtopping and collapse of dike sections of a certain length and hydraulic structures such as sluices. Some potential human mistakes in operation and maintenance of man-made elements were also considered. The study developed a computer program which was described as a "prototype", considering the necessity for further alteration. Afterwards this model and program were applied in Polder of Noord-Schuddeland (Vrouwenvelder, 1994).

Vrijling (1987) and Vrouwenvelder (1993 and 1994) made progress by examining the probabilities of different failure mechanisms focusing on overall rather than just independent behaviour. However, since the correlation between failure mechanisms has usually to be assessed on the basis of historical data which, as mentioned before, lack homogeneity, the correlation can only be determined with great uncertainty.

To provide a preliminary basis in practical design, the percentage contributions of different failure mechanisms to the overall failure were recommended by TAW (Technical Advisory Committee on Water Defences) (1989) as presented in Table 2.6. The percentage relations are further studied by TAW.

<table>
<thead>
<tr>
<th>failure mechanisms</th>
<th>determination of failure probability</th>
<th>percentage relation</th>
</tr>
</thead>
<tbody>
<tr>
<td>lifting the permeable layer, piping, micro-stability, bank erosion</td>
<td>deterministic</td>
<td>20%</td>
</tr>
<tr>
<td>overtopping and overflowing, macro-stability, hydraulic structure and construction</td>
<td>probabilistic</td>
<td>70%</td>
</tr>
<tr>
<td>others such as: muskrat, falling of trees, ship's collision</td>
<td>not possible</td>
<td>10%</td>
</tr>
<tr>
<td>overall failure</td>
<td>probabilistic</td>
<td>100%</td>
</tr>
</tbody>
</table>

The development process of a sand-dike breaching was studied by Visser (1994). For a sand-dike without any covering layer and given an initial breach, a breach could be expected of 40 meter in width within one hour. In the follow-up study was stated that breaching would continue further but the speed of the development in terms of the breach width would slow down.
Chapter 2

It was concluded by Boertien I that the present knowledge does not permit to predict whether a dike would breach when the water level exceeds the dike height. The Commission (MTPW, 1993) assumed that in case of overflowing, some polders may be flooded, and others may remain "dry" (not flooded). This reflects the difficulty in predicting under what conditions a dike overflowing will result in breaching and how large the breach can be.

Therefore two extremes resulting from dike overflowing should be considered: pessimistic (the dike would breach due to overflowing) and optimistic (the dike would not breach in the course of overflowing). The pessimistic extreme has been examined in several studies in the Netherlands (Delta Committee, 1962; MTPW, 1977; Vrouwenvelder and Wubs, 1989); no study included the optimistic extreme. The flood scenarios applied by Boertien I implicitly adopted the pessimistic extreme (all polders would be flooded) and the intermediate of the two extremes (one and three polders would be flooded). The foregoing studies can be summarized with respect to breaching in Table 2.7.

2.4.5. Other areas of concern

To promote sophisticated designs, the TAW assembled the readily available, but adverse expertise (including the studies reviewed above) into design guidelines for river dikes (TAW, 1989 and 1991). The design guidelines cover design, maintenance and management aspects and various failure mechanisms.

Table 2.7. A summary of studies on dike overflowing with and without breaching

<table>
<thead>
<tr>
<th>study</th>
<th>with breaching</th>
<th>intermediate cases</th>
<th>without breaching</th>
</tr>
</thead>
<tbody>
<tr>
<td>Delta Committee</td>
<td>x</td>
<td>being aware</td>
<td></td>
</tr>
<tr>
<td>Becht Commission</td>
<td>x</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vrouwenvelder and Wubs (1989)</td>
<td>x</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Boertien I</td>
<td>x</td>
<td>x</td>
<td></td>
</tr>
</tbody>
</table>

Bakker (1989) assessed the impact of long term sea level rise for the areas outside of the primary dikes of polders where many industries locate. In this study an ARC/INFO GIS was applied to merge the borders of polders with the land use in these areas in order to inventory the land use and further to assess the impact of the sea level rise. The
digitized land use map had a lower limit of 60000 m² for the polygon elements. It was the first time (in the Netherlands) that flood damage assessment was assisted by GIS. This lower limit is probably sufficient for the study at a macro level such as the impact of sea level rise; the newly developed digitized land use map in the Netherlands has a still lower limit for polygon elements.

In summary, practice in the Netherlands has made obvious progress in as determining the protection level on the basis of economic optimization as well as human life loss and introducing innovative dike design to reduce environmental impact of dike improvement. However, various issues still remain to be addressed.

2.5. Issues for further study

The above overview on the studies for flood protection reveals the following issues for further study:

1. Decision analysis in flood alleviation considering implementation timing and capacity under budget constraints. The optimal sequence of dike heightening should be determined on the basis of all possible sequences of dike heightening to maximize total net benefit. Due to the hydraulic interdependence of dike heightening, as soon as dikes are heightened, the hydraulic situation in the whole area should be re-simulated. Because of the usually very large combinatorial number of dike heightening sequences, this would be a very time-consuming process and a true optimization of heightening sequence is practically not feasible. The optimal sequence of implementing flood alleviation measures has not yet received sufficient attention.

2. Overall probability of failure mechanisms. The overall probability of flood depends on the probability of individual failure mechanisms and their correlations. At the present time, the knowledge of this overall failure probability is inadequate. Since the ultimate goal of flood alleviation is to reduce the overall expected flood damage, which is the probability-weighted flood damage of individual failure mechanisms, further studies should focus on (1) the probability of individual failure mechanisms, (2) the correlation between failure mechanisms, (3) the reduction of flood damages caused by individual mechanisms, especially those mechanisms estimated to bring larger contributions to the overall failure probability (refer to Table 2.6).

3. Dike height determination versus assurance of hydraulic and geotechnical stability. During the latest flood in the Netherlands, a lot of people had to be evacuated not because of the potential danger from the failure mechanisms associated with dike
height such as overflowing and overtopping, but from the failure mechanisms and instabilities associated with other dimensions of the dike body such as water load and soaking. The dike improvement program being undertaken in the Netherlands focuses mainly on dike strengthening. As the impermeability and stability of dikes improves, the dike height would become the most decisive factor in protecting polders from flooding.

4. Polder flooding induced by dike overflowing. The two most important failure mechanisms to consider in the design of the dike height are overflowing and overtopping. The Dutch river dikes are mostly covered by grass, clay and pavement, therefore one can expect that the development of the breach of a river dike is slower than that of a sand-dike. It is unlikely that a river dike can sustain itself if the dike overflowing is significant. Overflowing does not necessarily always develop to its ultimate stage, if the dike is covered with materials of good erosion-resistance, or if overflowing is not significant. Contingency measures by rescue teams standing by during flooding, as in 1995, may prevent overflowing at critical sites. There is no study yet that explicitly considers both overflowing and breaching in flood protection.

5. Improvement of the estimation of the occurrence frequency of flood events, flood damages, costs and impacts. In particular, inventory of potentially flooded property is time consuming, and involves great uncertainty. Proper utilization of the newly developed digitized land use maps for such an inventory can be helpful for decision analysis for flood alleviation with a large spatial coverage.

6. Uncertainty analysis. Various uncertainties in flood alleviation studies exist. Some are due to the inherently random nature of natural events and non-stationary conditions, some are due to insufficient knowledge such as failure mechanisms of structural measures. Others are due to insufficient information on such factors as property damages and economic investments for alleviation measures, or to the unpredictability of socioeconomic development and people’s expectations and valuations. Without analysis of these uncertainties, decision analysis is less robust.

Studies on the above issues are expected to improve substantially decision analysis for and decision making in flood alleviation. With the present knowledge, however, some are quite difficult to deal with. This study limits itself to the issues presented in the next sections.

2.6. Narrowing the gap between theory and practice

The civil engineering profession traditionally viewed itself as responsible for
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guaranteeing safety as much as possible. In fulfilling this responsibility, designers of
dikes and dams often incurred very high expenditures for designing and constructing
protection structures under unknown conditions. To avoid over-design or under-design
of hydraulic projects, the reduction of flood risk and damage should be traded off
against the cost of flood alleviation.

Two approaches can be followed to determine the required structural measures for flood
alleviation, namely, application of a normative protection level or a local acceptable
level based on a specific study. The application of a norm is based on the experience
of similar projects in the past and thus contains an implicit balancing of the reduction
in flood damage and the investment in protection measures. A normative protection level
is usually applied to everyday projects such as tunnels, bridges, or culverts. For a large
project of flood alleviation, the acceptable protection level should be investigated on the
basis of the local situation because of the cost-effectiveness of the investigation and the
requirement for the social and environmental consideration.

Flood alleviation should be regarded as a dynamic multiobjective decision process under
uncertainty. Firstly, flood alleviation aims at several objectives such as reducing flood
damages and human life losses, maximizing the efficiency of investment in flood
protection, and limiting the adverse social and environmental impacts of flood alleviation
measures. Secondly, flood alleviation at a large scale requires substantial investments
and the completion of flood alleviation measures may take years. Implementation timing
of protection measures plays an important role in flood alleviation due to, particularly,
the hydraulic interdependence of different alleviation measures. Thirdly, supporting
information for decision analysis for flood alleviation measures is subject to various
uncertainties. Budget restrictions and the consideration of the adverse social and
environmental impacts limit the decision space.

Analytical techniques for dynamic multiobjective decision analysis under uncertainty and
optimization have become available and applications in many fields have been reported.
These techniques are effective only if all the information needed for the analytical
models can be readily obtained. However, this is usually not the case with flood
alleviation. Decision analysis applications in flood alleviation are to be characterized as
partial optimization considering the difficulties in formulating complete flood alleviation
measures (strategies), in depicting hydraulic characteristics in hydraulically
interdependent floodplain reaches, in estimating supporting information for the decision
model, in quantifying various uncertainties, and in grasping people’s expectations to and
valuations of different objectives and "intangible" aspects.

The challenge in improving the support of decision making in flood alleviation is thus to narrow the gap between theory and real life practice. This study intends to narrow the gap by determining in particular the optimal flood protection level. The ultimate challenge in flood alleviation at the strategic level is to determine the acceptable level of flood protection and to allocate the limited public budgets efficiently to (and within) flood alleviation.

2.7. Focus of the study

The Dutch practices in dike improvement represent the state-of-the-art internationally and therefore this research focuses on the following several aspects of decision analysis for flood alleviation, which are derived from the review of this chapter:

(1) The complete alternatives for flood alleviation by means of dike heightening will be formulated, and the economically optimal alternative will be found through the risk assessment approach.

(2) Methods and procedures for the assessment will be developed for the river transition zone. The river transition zone is the most complicated area in a river basin as far as the estimation of flood magnitude and its occurrence frequency is concerned. In principle, the corresponding methods and procedures for a river regime zone and a tidal zone are more simple.

(3) In order to avoid the labour intensive inventory of the properties in polders (but still at a fairly accurate level), the newly developed digitized land use information will be applied and handled by a GIS to assist the assessment of tangible direct flood damages. The application of GIS offers a potential to incorporate land use zoning with structural measures and makes it possible to develop flood alleviation policy over a broader geographic coverage.

(4) The various uncertainties—for example, those in failure mechanisms of overflowing, in hydrologic uncertainty, in flood damage assessment and in dike improvement cost—in the decision analysis for flood alleviation will be quantified and their influence on decision analysis will be analyzed through a real world application.

In the next chapter, design support in river dike improvement will be discussed.
CHAPTER 3

DESIGN SUPPORT
IN RIVER DIKE IMPROVEMENT

3.1. Introduction

Flood alleviation is, mostly, stimulated by flood events and changing prevailing societal climates. When the society becomes increasingly aware of potential flood damages or experiences a flood disaster, the current flood protection will be questioned.

Decision analysis for flood alleviation requires basic information on two aspects: flood damages, and the costs and adverse impacts of flood alleviation measures. The expected flood damage reduction by alleviation measures is regarded as the benefit of the measures. The adverse impacts are the social and environmental impacts of flood protection works. The net benefit of alleviation measures is the difference between the reduced expected damage and the cost and adverse impact of alleviation measures.

The ultimate decisions in river dike improvement are whether, how, where and when to upgrade the flood protection. Actually these decisions are all connected with each other. For example, when decisions are made to upgrade flood protection, flood alleviation alternatives (structural and non-structural measures) should be considered, in conjunction with the question of how to upgrade. Further the net benefit of the options should be compared with the do-nothing situation. Since flood alleviation at different places and different times influences the net benefit, hence the issue of where and when to upgrade is also linked. Therefore, the set of decisions should be approached iteratively.

In this chapter, river dike improvement (Section 3.2), dike failure mechanisms (section 3.3) and composition of dike height (Section 3.4) will be introduced. Further, three sets of often used tools for the design of structural measures and the support of decision making in flood alleviation will be reviewed, namely, reliability analysis (Section 3.5), economic optimization (Section 3.6), and multiobjective evaluation (Section 3.7).
3.2. River dike improvement

In the design of river dike improvement or new river dikes, three levels of decision making can be distinguished: strategic, intermediate (or tactical) and operational levels. The different decision levels in dike improvement are illustrated in Table 3.1.

<table>
<thead>
<tr>
<th>decision making levels</th>
<th>design variables</th>
</tr>
</thead>
<tbody>
<tr>
<td>strategic level</td>
<td>type of alleviation measures, location, protection level and dike height</td>
</tr>
<tr>
<td>intermediate level</td>
<td>alignment, structure and cross-section, building technology</td>
</tr>
<tr>
<td>operational level</td>
<td>dimensioning (slope, base width), material selection</td>
</tr>
</tbody>
</table>

At the strategic level, decisions are made on what kinds of structural measures can be applied and how safe the floodplain should be (the protection level). The location and height of the dike associated with the protection level are determined. Design decisions at the intermediate level include: alignment, the determination of structure and cross-section to ensure hydraulic and geotechnical stability and the choice of construction technology. At the operational level the dimensions of the dike (slope and base width) and the construction materials are determined in detail and, if necessary, refined. In innovative design, construction materials and construction technology may prove to be strategic rather than purely operational or intermediate design variables.

A simplified cross section profile of a conventional dike can be sketched as in Fig. 3.1. A series of dike segments constitute a ring of dikes protecting the polder from the direct flooding danger from sea or river. In addition to these outer dikes, inner dikes segment some polders and have significant influence on the redistribution of flooding water in the polder in case the polder is flooded.

River dike improvement with the conventional dike profile usually has undesirable adverse impacts: houses may have to be demolished, unique landscapes may be affected, and the local community structure may be disrupted. Conventional dike construction falls short in reconciling the conflicting demands of safety and environmental quality. Limitation of adverse social and environmental impacts of dike improvement has therefore become a major concern. Innovative design concepts for river dike improvement were introduced to reconcile upgrading the flood protection and at the
Design support in river dike improvement

same time preserving the environment and protecting the unique landscape around the dikes.

Fig. 3.1. Simplified cross section profile of a conventional dike

A "smart" design, defined by the Becht Commission (MTPW, 1977), involves intensive research and advanced calculation methods to develop tailored design concepts and construction methods, which reduce adverse effects of dike improvement. Smart designs usually include location changes to avoid demolishing buildings on the existing dikes, and/or strengthening the existing dike by installing fixed constructions mainly underground (MTPW, 1993). These fixed constructions therefore have less adverse impact on the river's landscape and on natural and cultural values. Fixed constructions such as cofferdams, sheet piles and filter construction can improve the dike stability and prevent failure mechanisms such as piping (Walker et al., 1994). An overview of innovative design concepts is presented in Fig. 3.2. Conventional dike improvement may have to demolish houses (Fig. 3.2-a). A coffer dam (Fig. 3.2-b) and foundation supporters (Fig. 3.2-c) in contrast can improve the stability of a dike body of a given height, an impermeable filter (Fig. 3.2-d) can prevent piping, and a mobile free board (Fig. 3.2-e) can prevent overflowing and overtopping.

3.3. Failure mechanisms

Numerous studies of dam failure have indicated that overflowing and overtopping are major failure modes of earth- and rockfill dams (e.g. Lemperiere, 1993). Dike heightening is only one of the structural alternatives for flood alleviation and the actual
failure of a dike is dependent on many factors. For example, it has been observed in the Netherlands that dike breaching in the past occurred, due to seepage, almost exclusively at the site where the dike body is built on a sandy strip (Berendsen et al., 1994), since high water levels can relatively easily undermine the dike of this foundation. As the water resistance of dike body and foundation or covering materials improves, dike height becomes more and more important in flood alleviation. A complete introduction of all dike failure mechanisms is not included but can be found in Vrijling (1987) and TAW (1989).

Fig. 3.2. Some examples of smart design (source: Boertien I)

In the field of dikes and dams, the practice and history of failure shows that complete safety is unattainable and economically undesirable. Attention, therefore, should be given to all possible failure mechanisms of the construction under design.

The overall failure mechanism of a dike section can be described by a fault tree. Vrijling (1987) formulated a complete fault tree of a dike section which captures the various failure mechanisms. Based on the fault tree, a simplified fault tree of a dike section is presented in Fig. 3.3. This fault tree includes not only the physical mechanisms but also
Design support in river dike improvement

those associated with human errors and destruction.

![Fault tree of a dike section](image)

Fig. 3.3. Fault tree of a dike section

Different failure mechanisms can influence the functioning of each dimension in the cross section profile of a dike body differently. Table 3.2 illustrates such influences.

Since this study, as clarified in the preceding chapter, only focuses on dike heightening in flood alleviation, the failure mechanisms associated with dike height will be highlighted. These failure mechanisms are critical at the strategic level of decision making on dike heightening. Dike height determines the protection level corresponding to the maximum flows to be confined by the dike section. Other failure mechanisms and design variables which determine the hydraulic/geo-mechanical stability of the dike can be handled separately for alternative dike height designs. Some failure mechanisms such as piping can be controlled by inspection and maintenance or prevented by a wire mesh (e.g. animals digging holes). The most likely dike failure mechanisms associated with the dike height include overflowing, overtopping and dike subsidence.

Overflowing is a well known mechanism. If the water level at a dike (or a dam) exceeds the dike crest, water will overflow into the area protected by the dike and consequently flood may occur. Initiated by overflowing, soil penetration (for a soil dike), erosion, and
soaking of the dike may occur and result in dike *breaching* and flooding. Breaching can be regarded as the ultimate stage of overflowing.

**Table 3.2. Failure mechanisms versus dimensions of a dike profile**

<table>
<thead>
<tr>
<th>failure mechanism</th>
<th>dimensions of a cross section profile</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>crest height</td>
</tr>
<tr>
<td>overflowing</td>
<td>X</td>
</tr>
<tr>
<td>wave overtopping</td>
<td>X</td>
</tr>
<tr>
<td>soil subsidence</td>
<td>X</td>
</tr>
<tr>
<td>macro-stability</td>
<td>X</td>
</tr>
<tr>
<td>foundation failure</td>
<td>X</td>
</tr>
<tr>
<td>piping</td>
<td></td>
</tr>
<tr>
<td>bank erosion</td>
<td></td>
</tr>
<tr>
<td>micro-stability</td>
<td></td>
</tr>
<tr>
<td>human error</td>
<td>X</td>
</tr>
</tbody>
</table>

When the water level is lower than the dike crest and a dike is exposed to wave action, a wave run-up can induce some floods as well. This is known as a wave *overtopping*. In this case, the amount of water entering the polder is insignificant. Overtopping may, however, similar to overflowing, be followed by soil penetration, slope erosion and dike soaking, and ultimately result in breaching.

*Subsidence* of the crest of the dike or dam may occur due to settlement of the dike and/or subsoil. Settlement may also be caused by internal erosion or by oxidation of peat layers.

### 3.4. Composition of the design dike height

The design river dike height in different hydraulic regimes—given a design safety level—can be determined in a two step procedure, as recommended by TAW (1989 and 1991). The first step is to calculate the so-called design water level, and the second, to determine the design height of a river dike. The procedure differs for the three hydrological regimes.
Design support in river dike improvement

Step one: calculate the design water level

Tidal zone: determine the sea level which corresponds to the safety level from the probability curve of the sea level; calculate the water levels along the river in the tidal zone using the sea level as the hydrologic condition at the boundary (the hydrologic gauging station). The resulting water levels are the design water levels.

River regime: follows the same procedure as that for the tidal zone but takes upstream discharge as the boundary condition.

Transition zone: the procedure is more complicated than those for the tidal and the river regimes since in the river transition zone both the sea level and the river discharge determine the water level along the river. It goes as follows: firstly establish the relationship of the levels of the hydrologic variables between hydrologic boundaries and the sites of interest along the river (with a one-dimensional model of river flow); secondly extrapolate the probability curves of the hydrologic variables at boundaries and transfer them to the probability curves of water level at these sites (local water level) based on the relationship; thirdly determine the design water level at different sites along a river corresponding with the pre-determined safety level from the probability curves of local water levels.

Step two: calculate the design height of river dike

The design water level found in three hydraulic regimes should be adjusted due to the effects of a.o. the long term sea level rise, oscillation and wind gusts.

Finally the design height of the river dike should be determined mainly on the basis of the adjusted design water level plus a safety margin. The safety margin is taken in order to compensate, for the wave run-up, the settlement of a dike body (if it is an earth dike), the subsidence of the sub-soil under the dike body, and the uncertainty in hydraulic calculations.

The calculation of the adjusted water level differs also with hydraulic zones. The composition of the design dike height is illustrated in Fig. 3.4.

3.5. Reliability based design

In this section, methods to design protection works will be introduced at different levels of complexity. These methods can be extended to deal not only with the design and evaluation of protective structures but also with the performance of operation and
management schemes (Duckstein and Plate, 1987).

Fig. 3.4. The composition of a design dike height

Protection works were constructed, from early times on, for protecting against periodical floods of settlers in flood plains. In the past, after the flood record was broken, protection was upgraded to the highest water level which had been experienced in the past. Design based on the highest flood recorded can be described as a Level 0 approach (trial and error).

To understand the nature of floods and to know the chance of being flooded, hydrologic variables such as water level and discharge were systematically observed and analyzed since the end of the last century (e.g. Shahin, et al., 1993). It was found that the occurrence of extreme values of hydrologic variables could be described adequately in terms of frequency in accordance with the laws of probability. Decisions on flood protection since then were increasingly based the occurrence probability of floods and reliability-based or probabilistic methods were developed with increasing levels of sophistication.

**Reliability and failure**

Reliability of a system can be described as the probability that a system is in a satisfactory state (no failure). It is the complementary probability of risk (Hashimoto,
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et al., 1982). Failure is defined as the situation in which the structure is incapable of functioning to satisfy the desired objectives (Ganoulis, et al., 1991-a). One can interpret the failure of a system in different ways. The flood defence system of a polder may be defined as a failure if one dike section is overflowed. The failure may also be interpreted as expected flood damage exceeding a certain acceptable level.

Given a design standard (an acceptable level of risk), the safety evaluation of an existing structure and the determination of the dimensions of a new structure are always related to the structural response as defined by the failure mechanisms. To determine whether or not a system fails and consequently to design a system to satisfy the given design standard, two forces are to be considered, namely "load" and "resistance". For example, in case of a river dike used for flood protection, the "load" is the flood stage, duration, exposure and wind; and the "resistance" is the dike height, hydraulic and soil resistance to slope instability and erosion. For the overflowing failure mode of a dike, the "load" is the water level and the "resistance" is the dike crest height. Evidently, definitions of the load and resistance of a system are required for further discussion on the failure of a system. Possible interpretations of load and resistance for other kinds of projects such as bridge piers, water supply systems, underground excavation, water quality, waste management and recreation can be found in Duckstein et al. (1987).

Load and resistance determine the reliability of a structure. If, in a common practice, the combination of all loads is defined as a generalized "load" S, and the combination of all resistances as a generalized "resistance" R, then a failure occurs if and only if "S > R" (e.g. CIRIA, 1977; Duckstein, et al., 1987). The reliability function of the system can be written as "Z = R - S".

In many situations, both the load and the resistance of a system are to be considered as random variables. In reliability based design of structures under random loads four levels of design are generally distinguished (Duckstein and Plate, 1987). The common feature of the four design levels is that a decision variable, the design resistance R_d of a structure, is chosen in such a manner that the structure only fails if the load exceeds a critical load S_d; the difference in the four levels lies in the specifications of R_d and S_d, and in the data required for the design. The higher the level, the more data are required. However, the complexity of analytical tools does not necessarily vary in the same direction (Duckstein and Plate, 1987).

With different assumptions about the probability distribution of R and S, the reliability
function can be calculated at three different levels.

**Level 1 design**

*Level 1* design is based on the concept of a safety factor. If $S$ is the specified standard, or the calculated load due to (some percentage of ) the worst-case level, then it is necessary to dimension the structure so that $S \times \eta \leq R$, where $\eta$ indicates the safety factor (e.g. CIRIA, 1977; Duckstein and Plate, 1987; TAW, 1990). Hence, important is the choice of the load level and the safety factor (the higher the safer) from which the resistance can be deduced. The level one technique provides a method of checking whether a defined level of safety is satisfied in routine (everyday) design practice.

This conventional type of design has been questioned inside and outside of the engineering profession. First, the economic efficiency of meeting (some percentage of) the worst-case level is usually not known. Design standards based on this calculation may have been set without rational procedures and adequate data. That is, a standard may lead to over- or under-design for a specific situation. Furthermore, standards may not incorporate new knowledge or the latest technology and data. Finally, standards submerge information about occurrence frequency, the damage, costs, and benefits of alternative solutions and consequently bypass the discussion about acceptable risks and willingness-to-pay.

**Level 2 design**

*Level 2* design is based on a second moment analysis. If loads and resistances, as well as the parameters determining loads and resistances, are normally distributed, or if their distributions can be approximated by or transformed into a normal density function, then the probability distribution of failure of the structure also follows a normal distribution (Duckstein and Plate, 1987).

This technique has been applied extensively to various kinds of hydraulic structures (e.g. Kiureghian, et al., 1987; Lamberti, 1992); especially for uncertainty analysis. Application to uncertainty analysis will be discussed in Chapter 6.

**Level 3 design**

*Level 3* design is concerned with the determination of the probability of failure without a prior specification of the probability density distribution of resistance and load. All variables in the reliability function are expressed in terms of their full probability distribution functions and probabilities of failure are computed by the evaluation of the
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appropriated convolution integral. This implies that the distribution of both resistance and load is examined in terms of their own (different) characteristics. The most important assumption at this level is that resistance and load are non-correlated (e.g. CIRIA, 1977; Duckstein and Plate, 1987)

Vrijling and Bruinsma (1980) applied this technique to the joint probability density function of wave spectra and storm surge levels. Studies with this technique by means of integrating the joint probability of the load and resistance include: hydrologic risk evaluation in Tang (1980), risk assessment of a levee system due to various modes such as overtopping, boiling, slope sliding and wind wave erosion in Duckstein and Bogardi (1981). Warner and Kabaila (1969) calculated the distribution of resistance and load and Duckstein, et al. (1981) estimated the probabilities of failure events for mine flooding, all by means of Monte Carlo simulation.

Obviously, design at this level invariably requires multi-dimensional numerical integration or the use of Monte Carlo techniques. The theoretical and numerical complexities of these techniques may inhibit application in routine design. Nevertheless, there is considerable scope for the use of Level 3 techniques for checking the validity and accuracy of the simplified calculations at Level 2 and Level 1 done to analyze specific structures.

Because reliability specifications at Level 3 and 2 require probabilistic calculations, they are usually characterized as "probabilistic design" or "risk-based" approach at Level 3 and 2 in the literature. It is important to note that the calculation process in Level 1 through 3 design starts with the specification of some formally accepted probability of failure (respectively design resistance) of the designed structure. The calculation techniques at the above three levels accept these standards as a starting point.

In project design, over-design or under-design may occur due to uncertainties, and the consequences can be vastly different (Beard, 1994). On the other hand, the possibility of under- and over-design can increase greatly when the economic performance of projects designed is not examined. For example, if flood consequences are not integrated into the design process, flood alleviation projects would not lead an economic optimum (Zhou and Van der Heijden, 1994-a). Design techniques at Level 1 through 3 design are not applicable for the evaluation of the economic performance of protection works or effectiveness of non-structural measures. Therefore, in addition to these techniques, other approaches considering design consequences are required.
Level 4 design

Level 4 design explicitly considers the economic, social, environmental and other consequences of a design in "figures of merit" (FM), to derive a composite multidimensional performance measure (Duckstein and Plate, 1987). At this level, the risk is introduced as a figure of merit. FM is the criterion which rates the performance of the system. Whereas the first three design levels use an externally imposed design criterion such as the safety factor or the probability of failure as a performance measure, the level four approach does not specify a predetermined numerical value for the design. The design value is found by optimizing a criterion function such as minimizing cost, minimizing the probability of fatalities, maximizing net benefit, or maximize benefit cost ratio. Conflicting FM or criterion functions may be considered simultaneously in a multicriterion analysis (Duckstein and Plate, 1987).

For a flood alleviation project, the consequences can be interpreted as the reduction of both flood damage and cost of the project. In the case of flood alleviation of several polders, if the consequences of the project for each polder differ, each polder as a system may have a different safety level through Level 4 design.

Within the level four approach, further refinements can be and should be made to account for successive improvements towards the ultimate goal: capturing all causative events, considering all failure mechanisms and including in the trade-off construction and expected maintenance costs as well as all social, economic and environmental flood damages. A full-fledged Level 4 approach is only feasible, for protection works in flood alleviation in particular, if all causal factors, boundary conditions and failure mechanisms can be incorporates into multivariate reliability and inundation models, which capture the full range of structural and non-structural measures (Pols, 1995). The problem of placing a monetary value on human life and other intangible damages cannot yet be dealt with satisfactorily. Consensus on values in risk assessment and acceptance would also be necessary.

The economically optimal level of risk can only be determined endogenously if all these conditions are met by including uncertainties and trading off costs and damage reduction. It should be noted that the safety level then would vary among inundation protection areas depending on differences in construction costs and expected damage reduction. A uniform safety level may be considered desirable by policy makers, in spite of individual or group differences in risk perception and acceptance, so as to prevent preferential treatment (Pols, 1995).
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The design concept of the level 4 approach can be further extended to higher levels. In the context of flood alleviation, the estimation of expected flood damage in a level 4 approach tends to be based upon the current state and autonomous development of the flood protection area; it does not fully capture the potential for damage reduction by land use planning and control and other organizational, financial, and regulatory measures. The level 5 approach would integrate structural and non-structural measures for flood alleviation, accounting for interdependencies, and fully incorporate the possible contributions of these measures to utilize flood-prone areas for beneficial use and damage potential reduction. The level 6 approach can be characterised as "risk balancing": the balancing of various kinds of risks to achieve an equitable distribution of exposure to various risks and an efficient allocation of resources in risk reduction efforts. The work of Rasmussen (1975) in the field of nuclear energy can be considered as a pioneering effort towards a level 6 approach.

Design approaches from Level 1 to Level 6 can be illustrated with Fig. 3.5 (Pols, 1995).

3.6. Analytical techniques for economic optimization

When the protection level is not given, economic optimization might be helpful in finding one. A wide range of optimization problems in flood alleviation is involved in determining for example capacities of reservoirs and channels, investment in flood warning and proofing, flood insurance premiums, and patterns of land use.

Decision analysis on flood alleviation has been categorized according to the alleviation measures (structural or non-structural, or both) as well as according to the optimization techniques employed. A detailed review on the decision analysis for different kinds of alleviation measures (structural and non-structural) can be found in Thampapillai and Musgrave (1985). For each kind of measure three types of analytical techniques are available, namely, discrete enumeration of cost & benefits, classical optimization and dynamic programming. Yeh (1985) and Wurbs (1993) provided extensive references on the use of linear, non-linear and dynamic programming in reservoir-system analysis.
3.5. Illustration of design tools at Level 1 to Level 6

Discrete enumeration of cost & benefits is the technique to compute the expected total net benefits on the basis of discrete pre-specified values of the decision variables such as dike height. The values of the decision variables which coincide with the highest expected total net benefit are chosen. An extension of this technique is the computation of benefits and costs in relation to pre-specified design and management strategies. General introductions for structural measures can be found in e.g. Eckstein (1958), Linsley and Franzini (1972), and Dasgupta and Pearce (1973); applications to flood warning systems in Day, et al. (1969), to both structural measures and floodplain zoning in Whipple (1969). This technique has also been used by the Becht and Boertien Commissions (MTPW, 1977 and 1993).

The major shortcoming of this technique is the inclusion of only a limited number of design and/or management options in the evaluation. Hence the strategy identified in such an evaluation may often not coincide with the true optimum of a net benefit function.
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Classical optimization overcomes this shortcoming by optimizing a function rather than a number of preselected options. However, the choice of an optimal decision depends on the algebraic nature of the objective function and could become difficult in the event of multiple optima. The literature related to water resources management in general and applications to reservoirs in particular is extensive. The earlier applications of this technique to optimize the polder dikes' height in the tidal zone can be credited to Van Dantzig (1956). Recent applications include Bennis and Assaf (1994) for critical water level prediction. Linear programming is an often used optimization technique. It identifies an optimal strategy by selecting the best combinations of activities. The various design and management options represent the activities. The linear constraints are conjunctive constraints on combinations of design and management options. The constraints implicitly define an infinite set of alternative combinations of design and management options. However, the basic assumptions of linear programming, namely linearity and continuity, which imply convexity and additivity, could restrict economic and engineering analysis. Applications for reservoirs can be found in Windsor (1975), and Crawley and Dandy (1993).

An optimal sequence of decisions is usually needed in the event of inter-temporal planning over the various floodplain reaches. Dynamic programming identifies such an optimal sequence of decisions. Expected benefits of flood losses are optimized, in terms of the decision variables, subject to a set of constraints. Often these constraints operate as state variables. That is, the values that decision variables (design and management options) could take are conditioned by the values taken by the state variables. These state variables also serve as a linkage between successive time periods and/or floodplain reaches. Applications of this technique to reservoir (operation) decisions are reported by Hall and Buras (1961), Burton, et al. (1963), Lucke (1976), Allen and Bridgeman (1986) and Ji et al. (1994).

Thampapillai and Musgrave (1985) concluded on structural and non-structural measures for flood alleviation that real optimization has not yet been achieved due to the exclusive consideration of one type of measure, or due to the limited number of the combinations of both types of measures.

3.7. Towards multiobjective evaluation

The above mentioned analytical techniques for decision analysis have successfully
addressed many flood alleviation problems where only the economic aspects of flood alleviation were considered. In the economic evaluation of a public work, the present value of the cost of constructing and maintaining protection works must be less than the discounted stream of expected benefits: the difference between the flood damages anticipated with or without intervention. As the issue of human life losses and later in the 1970’s the concern of adverse environmental impacts of flood control works emerged, the estimation of "costs" and "benefits" became more and more comprehensive, and alternative approaches and methods were developed to overcome these difficulties.

It is generally recommended that the loss of human life should be examined in a separate analysis. For example, Lave et al. (1990) stated that distinctions should be made for dams which may result in great human life losses in case of failure and those which may result in less losses of life. If no life is at risk, the safety level of a dam can be simply determined by the cost-benefit trade-off.

River basin management for the purposes of flood alleviation is emerging in many countries. Before early 1980’s, much of this work has been carried out with little regard to the impact on the river environment and its associated aquatic community. However, the concern for ecologic resources increased since 1970’s (Swales, 1982) due to the need to pursue both safety and environmental quality. In recent years, considerable advances have been made in the theory and measurement of the economic value of environmental assets. Since changes in environmental quality are part of the consequences of a project, its economic values are to be considered when conducting cost-benefit analysis of policies and programs which affect environmental quality (e.g. Bergstrom, 1990).

Economically and environmentally sound flood protection works are the ultimate goal that many designers want to achieve. For example, James, et al. (1978) demonstrated a procedure to formulate the feasible urban flood control alternatives, by screening the economical feasible alternatives on the basis of criteria of social sustainability and contribution to ecologic and community well-being. Reeve and Bettess (1990) identified environmentally acceptable channel features, such as shallow-water berm, island and meanders/channel bends, which may incur some hydraulic problems such as limiting discharge capacity.

Most often, social and environmental objectives, if indeed they are specified, are not quantified in any units of measurement that could directly relate to the consideration of
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... economics and loss of human life. Various authors in environmental economy (e.g. Freeman, 1979; Bergstrom, 1990), ecology (e.g. Helliwell, 1972; Green and Tunstall, 1991), and water resources management (e.g. Bodington, 1993) recommend to evaluate environmental resources in economic terms when the loss or preservation of environmental resources is involved. The principal argument made for such evaluation is that, when all other consequences of the decision, except for the environmental impacts, are compared in monetary terms, the latter will be treated as of zero value (Green and Tunstall, 1991) or result in difficulty of comparison. The evaluation of environmental resources is usually based on public surveys of willingness to pay (e.g. James, et al., 1978; Green and Tunstall, 1991).

Although it is not widely agreed that economic evaluation of human life losses and environmental properties is desirable (e.g. Coker and Richards, 1992), there is no question that human life losses, flood damages to environment and environmental impacts of flood alleviation measures, especially those of structural measures, should be taken into consideration in decision analysis for flood alleviation.

To incorporate objectives of different nature, multiobjective evaluation is required. Multiobjective decision analysis techniques have been widely applied in water resources engineering as well as in other fields. For example, these techniques have been introduced to water resource systems planning and analysis (Loucks, et al., 1981; Mays and Tung, 1992), applied for watershed resources management (Tecle, 1992) and for ground water contamination management (Shafike et al., 1992).

3.8. Conclusions

Design techniques for flood alleviation measures at different levels of complexity have been reviewed and introduced in this chapter. Economic optimization is often applied, when the level of optimal flood protection or the spillway capacity of a reservoir in flood alleviation, has to be determined for a specific project. Multiobjective evaluation techniques should be used for objectives of different nature such as human life losses, flood damages to economic and environmental assets.

Given the importance of dike improvement in flood alleviation in the Netherlands, a decision framework for river dike improvement in the river transition zone will be further developed in the next chapter.
Part II

Theory
CHAPTER 4

RISK ASSESSMENT
IN RIVER DIKE IMPROVEMENT
— FRAMEWORK AND FORMULATION

4.1. Introduction

In this chapter, a decision framework for river dike improvement in the transition zone is developed. The transition zone poses a particular challenge for planning due to (hydraulic) interdependence of different alleviation solutions and the need to combine boundary conditions. As discussed in previous chapters, dike heightening can be seen as one of the possible measures for flood alleviation.

Planning for flood alleviation is characterized by multiple objectives and uncertainty and requires a dynamic approach to deal with changes of the system over time and phasing of the implementation, which requires large investment costs and a substantial implementation capacity.

In this chapter a practical approach for the total planning problem is proposed which contains two subsequent parts, namely:

- optimization of target protection levels for a given physical and socioeconomic system; and
- a dynamic optimization, consisting of phasing of the implementation of these targets and a periodic updating of targets.

The operationalization of risk assessment in this dissertation and the application in the subsequent chapters mainly focus then on the first step.

The formulation of the framework in the present chapter has been subdivided into three parts, namely, identification of the scope of the decision analysis process by briefly describing the relevant components and trade-offs (Section 4.2); general formulation of the decision analysis problem, emphasizing the trade offs and inter-relationships which complicate the solution of the problem (Section 4.3); and step by step formulation of a practical solution (Section 4.4).
4.2. Scope of the decision analysis process

4.2.1. Position of dike heightening in overall risk assessment

In the previous chapter an overview was given of risk assessment focusing on flood alleviation. Figure 4.1 places flood and inundation by dike failure, through overflowing and breaching, in the broader context of societal risks.

![Diagram of societal risks due to flood and inundation in the tidal zone, the transition zone, the river zone, other causes such as traffic, industry, war, ... leading to dike failures or other causes through overflowing & breaching or other failure mechanisms.]

**Fig. 4.1.** Position of dike overflowing and breaching in overall risk assessment

Different geographically zones have different meteorological, hydrological and hydraulic processes determining critical flood conditions. The river transition zone, influenced by up- and downstream boundary conditions, involves two boundary conditions which pose a challenge for risk assessment.

The present study focuses on the operationalization of the risk assessment approach in flood alleviation, in particular by means of dike heightening.

As discussed earlier, the actual failure of a dike is dependent on many factors but the height of the dike can be seen as an important factor in this total process. This study focuses on the determination of dike design height in response to the meteorological, hydrological and hydraulic conditions which can be expressed by a design water level in front of the dike. It is then assumed that other design requirements (e.g. preventing
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structural instability) are sufficiently met.

Two extremes of dike overflowing are considered: overflowing (where the existing dikes remain intact) and breaching (where the existing dikes are washed away for a certain length and depth). Dike height will be a more important dimension of the dike cross-section for overflowing than for breaching. Duration of flooding conditions will be more important for breaching. Both mechanisms determine strongly the volume of water entering the protected areas and the resulting damage from a particular critical flood condition.

Flooding has impacts on society in different ways, and involves economic, sociopsychological and environmental effects on individuals, groups and society as a whole.

4.2.2. Components in the decision analysis for dike heightening

The decision analysis process for dike heightening consists of a number of steps which are presented in the flow diagram in Fig. 4.2.

Fig. 4.2. Decision analysis process for dike heightening
For the existing river schematization (dike height and river cross-section profile), the inundation level in polders (or flood plains) can be simulated and the exceedance probability for each polder can be estimated. The transition zone is characterized by two boundary conditions (upstream and downstream) which have to be combined into a single occurrence probability distribution for each site in the transition zone. With the inundation depths and their occurrence probabilities, the expected flood damage can be further assessed. Any dike heightening changes the existing river schematization, thus for each alternative of dike heightening, the inundation depths and their occurrence probabilities, and the expected flood damage should be recalculated accordingly. This implies that it is only feasible to evaluate a finite number of alternatives. An alternative in this sense is defined as a set of locally connected dike heightening measures in a system of polders. The difference between the expected flood damage before and after dike heightening stands for the expected damage reduction and is usually regarded as the benefit of dike heightening. A decision on dike heightening should be made by comparing the net benefits of a set of dike heightening alternatives.

4.2.3. Types of uncertainty and role in the decision analysis

Uncertainty and dealing with uncertainty constitute a major component of the decision framework for flood alleviation. Attempts have been made by different authors to categorize uncertainty into different classes. A distinction is usually made between an inherent or intrinsic, model and parameter uncertainty (e.g. Mays, 1979). Inherent uncertainty associates with the stochastic process which can be considered an act of God or nature and will probably never be completely explained by any model. Model uncertainty has to do with the simplification in modelling the nature system, while parameter uncertainty is involved due to limited observation, deficient measurement techniques, or less effectiveness of model calibration. Other classifications of uncertainty, which are less widely used in water resources engineering, have been found in the literature. Rowe (1977) referred inherent and modelling uncertainty as "descriptive" uncertainty. Bernier (1987-a) referred modelling and measurement uncertainty as "technological" uncertainty.

All types of uncertainty exist in the decision analysis in flood alleviation; there is uncertainty associated with most of the design and evaluation parameters of the decision model on dike heightening in the transition zone. These uncertainties can be briefly described as follows:

- Inherent uncertainty on the meteorological occurrence of flooding situations: it will
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probably never be possible to fully predict the occurrence of floods and certainly not over prolonged periods of time.

- In the transition zone the combined influence of two boundaries has to be considered to determine the flooding probability. Since the probability distribution of the inundation level in a polder depends on the joint probability distribution of the boundary conditions, the extrapolation at boundaries and lack of information on this joint probability introduce an uncertainty to the probability distribution of the inundation level in the polder.

- Several uncertainties are involved in determining the physical process of flooding. At present the lack of knowledge on the actual process of flooding due to overflowing, with or without breaching, is one of the major sources of uncertainty.

- Flood damage assessment requires reliable information and effective assessment models. The information includes the nature and the quantity of the properties in polders and the socioeconomic linkage of the economic units in the polder with their clients, suppliers and employees. Insufficient information and ineffective assessment methods bring uncertainties into the flood damage assessment.

- Similarly, there is also uncertainty in the estimation of the cost and adverse impact of dike heightening. The perception and/or valuation on the non-monetary aspects can also change due to the changing societal climate, thus giving rise to uncertainty.

The various uncertainties require to consider the many options and interrelations, and therefore strongly increase the dimensionality of the decision analysis problem. Uncertainty has further the effect of diffusing the decision analysis process by decreasing the certainty of the statement about the differences between alternatives.

4.2.4. Interdependence in the transition zone

The flow in the transition zone is determined by an upstream flow boundary (river zone) and a downstream level boundary causing backwater effects. For flat delta regions, such as a large part of the Netherlands, this backwater effect exists for an extended area with a pronounced effect at the downstream boundary and a gradually diminishing effect in the upstream direction.

The extreme potential flooding conditions at the two boundaries have distinct characteristics with different influences on flooding. Extreme high water level at the sea boundary will last for about half a day while an extreme river flood wave will take 2
or 3 weeks to develop and recede. The hydraulic situation at any point in the transition zone will depend on the boundary conditions and can be determined by a simulation of water movement in the zone based on the boundary conditions.

Due to the particular hydraulic condition for the transition zone, the flooding at any site in the zone will depend on the hydraulics in the total zone and inversely. Changes in the hydraulic flow conditions, such as changes in the river channel and changes in the dike system, will influence the flow pattern and thus in turn the flooding.

A change in flood protection (dike heightening) for one polder will therefore not only change the flooding for the particular polder but also for the other polders. The different alternatives for flood protection are interdependent. This strongly increases the dimensionality of the decision problem as many combinations have to be considered.

4.2.5. Components in the decision analysis for dike heightening

Damage-probability relationship and expected damage

The basis for the decision analysis for the flood alleviation problem consists of a damage-probability relationship which is used to evaluate the effect of dike improvements with respect to the existing situation. A typical form of the relationship is presented in Fig. 4.3.

The relationship is based on a set of sample points {damage, exceedance probability} which are derived from monitoring, simulation and/or evaluation. When the exceedance probability of hydrological variables such as river water level is less than the present protection level \( P_0 \), flooding occurs with flood damages. The more severe the flood event, the more flood damage. This is presented in the left part of Fig. 4.3.

Dike heightening upgrades the present protection level \( P_0 \) to \( P_1 \). A new set of sample points of damage-probability can be derived after dike heightening. It should, however, be noted that dike heightening does not only reduce the exceedance probability of the polder, but can also result in changes in the sample points. Therefore the flood damages of a particular exceedance probability before and after dike heightening can be different, as indicated in the right part of Fig. 4.3.
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Fig. 4.3. Flood damage-exceedance probability relationship curve

There are usually two approaches for the specification of dike improvement, namely:
- a dike improvement is selected which reduces the existing $P_0$ to the pre-specified safety level $P_i$; and
- optimization of damage reduction versus the cost of dike improvement: different dike improvements can be characterised by different exceedance probabilities $P_i$; the $P_i$ upgraded by dike improvement which maximizes net benefits is selected.

As argued in Chapter 2, this second approach is followed in this study.

If $D(p, P_0)$ and $D'(p, P_i)$ respectively represent the damage-probability relationship for the existing and mitigated situation, then expected benefits can be expressed as:

$$B(P_i, P_0) = \int_0^1 D(p, P_i) \, dp - \int_0^1 D'(p, P_0) \, dp$$ (4.1)

The above represents a standard approach which incorporates the inherent uncertainty in the flood control problem and which can be directly applied to relatively simple situations such as the design of a levee in the river zone. As discussed in Section 4.2.3 the situation in the transition zone is more complex and considerable uncertainty is associated with the estimation of both the occurrence probability and the flooding process leading to damage. Those uncertainties can be interpreted as measurement
uncertainties on the parameters of the expected damage as outlined above, which deals essentially with the inherent uncertainty in the decision problem.

**Flood occurrence probability**

The occurrence probability of a particular hydraulic condition leading to an associated flooding condition at different sites (polders) is determined by the joint probability of the occurrence of the particular upstream and downstream boundary condition. More than one combination of boundary conditions may lead to the same volume of water entering into a particular polder. A mapping of the water volumes in a polder for the different possible combinations of the boundary conditions is therefore required. Figure 4.4 illustrates the concept.

![Diagram with discharge at upstream boundary and sea level, showing contours of equal flood volume in a polder.](image)

**Fig. 4.4. Mapping of flood volume in a polder in function of boundary conditions**

Contours of equal flood volumes of water \( \{W_j\} \) can be determined for a given protection for polder \( j \) from a sample set of flooding volumes derived (by simulation) from different combinations of the boundary conditions. The exceedance probability for a particular \( W_j \) can then be determined by integrating the joint probability distribution over the area above the contour line of \( W_j \) in the coordinate of the two axes representing two boundary conditions.
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Considerable uncertainty is associated with the extrapolation towards the extremes for each of the boundary conditions. The joint probability of occurrence goes a step further and incorporates the product of those uncertainties. Not only the annual joint occurrence is important but more in particular the timing in the order of days during the year. Further study of potential correlation would probably require a detailed study of (regional) meteorological conditions giving rise to extreme rainfall on the upper catchment and those creating storm surges at sea, and the likelihood that they jointly occur or even can occur. Details on the derivation of flood exceedance probability are presented in Chapter 5.

Failure mechanisms and damage assessment

As described in Chapter 2, flood protection in the Netherlands is organized in the form of polders, each of which represents an individual area protected by a ring of dikes. An outer and inner dike section is considered. The outer portion protects the polder from the direct flooding danger from sea or river, the inner dike sections segment the total region and contain the damage as much as possible to the individual polders. The individual polders can thus be regarded as the basic units for flooding and associated damage computations.

This segmentation and ring shaped dike defense system have particular consequences for damage computations: flood water entering the polder will ultimately fill the polder up to the lowest dike section in the ring. This constitutes the maximum possible inundation level and thus also the maximum damage; additional flooding water will discharge to a neighbouring polder or back up into the river. Upgrading (heightening) of the dike system for a polder from \( P_0 \) to an improved protection level \( P_1 \) will then have two consequences, namely:

- no flooding up to protection level \( P_1 \); and
- potentially increase inundation level associated with the increased lowest crest level of the ring dike system.

The actual failure of an earth dike is a complex process. The amount of water entering into the protected area is influenced by the construction characteristics as well as the duration of the high water level condition. Breaching, as described in Chapter 3, is the ultimate stage of overflowing, therefore the main uncertainty is whether the overflowing caused by a flood event will result in the ultimate stage. With respect to the amount of water entering into the polder two extremes can be considered for the failure
mechanism, namely:
- overflowing and no breaching of the dike: in this case the structure is considered to be sufficiently stable to withstand the scouring action of the flowing water and only flow over the dike is entering into the polder; and
- overflow and breaching: in this case a breach is made by the overflowing water.

The speed with which breaching occurs and its final size is uncertain. In previous flood control studies (e.g. Boertien 1) it has been assumed that the duration of the high water level condition in combination with breaching will be sufficient to fill the polder completely. Example computations readily show that it would take in the order of a few days to completely fill an average polder in the Netherlands. Flood conditions arising in the upstream river zone have then certainly sufficient duration to meet this requirement. Flooding conditions originating from high level at sea have a much shorter duration and a partly filled polder could be considered. Sufficient knowledge on the breaching process and associated flooding of polders is lacking; in most of the previous flood control studies in the Netherlands a complete flooding of polders was assumed.

In this study the overflowing case and overflowing with breaching will both be considered to test the sensitivity of the results for the two failure mechanisms. A complete flooding will then be assumed for all flooding events in the case of breaching.

The damage-probability relationships for the overflowing but no breaching case and the overflowing and breaching case are respectively presented in Fig. 4.5 and Fig. 4.6.

In the situation of overflowing only, polders will be fully flooded usually only when the flood is extremely large. As soon as the polder is fully flooded, flood damages remain constant, which is indicated by the horizontal part of the damage-probability curve in Fig. 4.5. If the lowest dike is heightened, overflowing may result in an inundation up to the new lowest dike height, and may consequently cause more flood damage than before dike heightening.

In the case of overflowing and breaching, as soon as the exceedance probability is less than the protection level, i.e. the water level exceeds the lowest height of dikes, the polder will ultimately be fully inundated up to the lowest dike height under extreme natural conditions. Similar to the situation of overflowing only, the flood damage after dike heightening can be larger than before. The difference in flood damage before and after dike heightening is illustrated in Fig. 4.6.
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Fig. 4.5. Damage curve for overflowing only

Fig. 4.6. Damage curve for breaching

**Flood alleviation measures**

As mentioned in previous sections dike heightening forms only one of a set of potential measures for flood alleviation. Lately, stimulated by environmental concerns a number of alternative designs have been added to the set. Dike height remains however a strategic design variable.

For the purpose of decision analysis for dike heightening the outer dike stretch is subdivided into sections for which the difference between the existing situation and a desired dike height is evaluated. The costs for upgrading are evaluated for each of those sections; the water level in front of the section is checked and relevant overflowing is computed. The desired level is determined for the whole of each individual polder.

In this study no upgrading of inner dike sections in combination with outer dike sections has been analyzed. Such upgrading of inner dike sections may get special attention if extra protection for a particular sensitive sub-polder is desired.

The expected damage in the existing situation serves as the reference situation to evaluate flood alleviation measures. With specific measures for flood alleviation, the cost of design, construction and maintenance can be relatively easily estimated.
Net benefits - selection of an optimal safety level

As mentioned above two approaches can be followed to determine the required dike improvement, namely, application of a normative safety level or optimization of net benefits. The application of a norm is based on the experience of similar projects in the past and thus contains an implicit balancing of benefits and costs. The conceptual origin for application of a norm does therefore not differ from an optimization of net benefits, it merely serves as a shortcut to avoid a more extensive analysis and evaluation of costs and benefits in each case. The assumption in applying such a norm is then that the previous situation also applies to the present problem at hand. The complexity of the protection problem in the transition zone as well as the diversity of values to be protected warrant a full optimization of the protection level. The present situation of different protection levels for the 53 polders in the Netherlands, which can be seen as the result of a long series of upgrades corresponding to a need for protection, further illustrates the need for a tailored approach to evaluate protection levels.

A typical pattern of the expected flood damage and the cost of improved protection is presented in Fig. 4.7. Similar conceptual figures have been presented in many papers (e.g. Van Dantzig and Kriens, 1960; Bernier, 1987-b; Vrijling et al., 1993).

The damage reduction can be expressed as the present value of a future stream of expected annual benefits (damage reduction) $B(P_t)$ for a particular protection level (exceedance probability) $P_t$.

The damage reduction curve shows decreasing returns to scale for improving protection levels. The cost curve exhibits increasing costs to scale: this can be derived from a cost function which is approximately linear in function of dike height, and the nature of the probability density function for flood peaks. The net benefit curve can be derived from the curves of cost and damage. A maximum for the net benefits can be observed corresponding to the optimal protection level $P_t$.

The optimization of the protection level is based on the value to be protected, the physical status of the river basin and cost for dike construction at a particular moment in time. Changes can occur in:

- the protected value resulting e.g. from increasing population and economic activity;
- the basin’s physical system such as deforestation, urbanization, and drainage
improvements which may increase (or decrease) flood peaks; and
- the costs for dike construction, e.g. due to environmental considerations.

Fig. 4.7. Cost and damage versus protection level

In the long term the optimal protection level should be considered as time dependent.

Multiple objectives

Flooding causes various socio-psychological, environmental and economic damages, and human life losses. Relevant objectives include: economic net benefits, income distribution, protection of human life, social stability and environmental concerns. Objectives expressing those aspects should be included in the evaluation of flood control measures. Such objectives can be incorporated in a multiobjective analysis of the flood alleviation problem, generalizing the traditional benefit-cost analysis which focuses on the estimation of expected costs and benefits in terms of national income.

Dike heightening does not only involve substantial economic investment in design, construction and maintenance, but also brings adverse social and environmental impacts.
The different damages and impacts and associated objectives are of a different nature and require different measuring units. In the framework of multiobjective analysis a total value, which is built up from the contributions from the individual objectives, of a particular flood alleviation strategy can be conceived. The selection of an optimal protection level as indicated in the previous section should then be based on the optimal net benefit.

**Implementation programming**

Implementation of a certain level of protection, if required for a fairly large area, normally requires substantial investments and constitutes a program, the implementation of which requires an extensive period of time. Phasing of protection improvements (dike heightening) is therefore an important part of the implementation. For the transition zone holds further that implementation of a particular dike improvement will alter the hydraulic conditions and therefore also the expected flood damage in the total set of polders. Therefore different phasing of dike heightening will result in a different present value of the expected flood damage reduction and different investment costs.

Combining the above observations and the conclusion in this section, the optimal protection levels may be seen as targets which should be achieved over time. As discussed above the optimal protection level itself should be considered as time varying in the long term, therefore the targets will also vary with time. In practice this will require an update of the targets at regular time intervals.

**4.3. Formulation of the flood alleviation problem**

**4.3.1. General**

The flood alleviation decision analysis problem can be formulated as an optimization problem which is characterized by a large dimensionality (large set of decisions in space and time), interdependence between alternative solutions, non-linear relationships and large uncertainties on both the process description and the parameter evaluation. Solution of this highly complex problem in a closed mathematical format is complicated and not feasible in practice; a best approximation thus has to be found (Zhou et al., 1995-a). The dominant characteristics of the problem should be used to decompose the total problem into sub-problems which can be handled separately. The total decision problem can be
decomposed into three main parts, namely:
- determination of an optimal protection level, ignoring implementation constraints as well as long term updates related to system changes;
- scheduling of implementation; and
- long term adjustments.

Using these components the flood alleviation planning process can be represented as in the diagram in Fig. 4.8.

![Diagram showing the process of decision and implementation of flood alleviation measures](image)

Fig. 4.8. Process of decision and implementation of flood alleviation measures

Based on the inventory of the flood control system (physical and socioeconomic characteristics), flood alleviation programs can be planned. The planned flood alleviation measures (projects) will be implemented gradually in the coming years to reach the target protection level. Since the flood control system changes with time, after a certain time period, the flood alleviation planning should be reexamined and, if necessary, the target protection level should be adjusted, and another round of effort of implementation will be followed. With reference to the approximation of the time dependant aspects, the above sketched approach to the solution can be called quasi-dynamic.

The three planning sub-problems are formulated below.

4.3.2. Optimal protection level

The decision analysis situation is characterized by objectives, attributes, decision variables and constraints. The practical aim of a multiobjective analysis is to identify
and enable trade-offs among conflicting non-commensurable objectives such that the best compromise solution can be selected through a political process. Nevertheless, it is worthwhile to find the solution by optimization.

**Objective function**
The flood alleviation problem requires to consider multiple objectives, such as maximizing the expected economic damage reduction, maximizing the expected human life loss reduction, maximizing the expected social damage reduction, maximizing the expected environmental damage reduction, minimizing the (construction and maintenance) cost of dike heightening, minimizing the social impact of dike heightening, and minimizing the environmental impact of dike heightening. The assumption that the individual objectives are separable and can be expressed in terms of expected value is rather strong but often made in practice, therefore it is used here for the formulation of the decision model. If the additional assumption is made that within a polder only the dike height is relevant for flood alleviation, the objective function is a function of the continuous variable termed protection level associated with dike heights of the polder and can be given as:

$$
\max \left\{ \sum_{j=1}^{n} B_e^j(P_i^j, P_o^j), \sum_{j=1}^{n} B_h^j(P_i^j, P_o^j), \sum_{j=1}^{n} B_s^j(P_i^j, P_o^j), \sum_{j=1}^{n} B_m^j(P_i^j, P_o^j), \right. \\
- \sum_{j=1}^{n} C_e^j(P_i^j, P_o^j), - \sum_{j=1}^{n} C_h^j(P_i^j, P_o^j), - \sum_{j=1}^{n} C_s^j(P_i^j, P_o^j), \left. \right\} 
$$

(4.2)

with:

- $n$: the number of polders;
- $P_o^j$: the present protection level $P_o$ (exceedance probability) for polder $j$ for each of the $n$ polders;
- $P_i^j$: the proposed protection level $P_i$ for polder $j$;
- $B_e^j(P_i^j, P_o^j)$: present value of the stream of expected future economic benefits (damage reduction) for polder $j$ for which protection level is proposed to upgrade from $P_o^j$ to $P_i^j$;
- $B_h^j(P_i^j, P_o^j)$: for the reduction of the loss of human life, similar to $B_e^j$;
- $B_s^j(P_i^j, P_o^j)$: for the reduction of the social damage, similar to $B_e^j$;
- $B_m^j(P_i^j, P_o^j)$: for the reduction of the environmental damage, similar to $B_e^j$;
- $C_e^j(P_i^j, P_o^j)$: the expected value of the cost for implementation of dike heightening measure to upgrade to the protection level $P_i^j$;
- $C_s^j(P_i^j, P_o^j)$: for the social impact, similar to $C_e^j$;
- $C_m^j(P_i^j, P_o^j)$: for the environmental impact, similar to $C_e^j$.
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**Constraints**

**Physical flooding process**
The protection level for polder \( j \) \( \{P_i^j, \ i=1, \ldots\} \) is a function not only of the measure taken for the polder but also of the measures taken for the other polders. The combined effect of upstream and downstream boundary conditions requires further to consider the joint probability of occurrence. The relationship between flood volumes \( W \) and their exceedance probability \( P \) for a particular polder \( j \) is a function of the protection level in all the polders as well as the boundary conditions and the river configuration, and can be expressed as:

\[
\forall \ j \ \{W, P\}_j = f (P_i^j, \text{sea level, river discharge, river configuration}) \quad (4.3)
\]

in which: \( \{W, P\}_j \) represents the sample point of the flood volume-exceedance probability relationship for polder \( j \); \( P_i^j \) denotes the protection levels of all polders.

**Flood damage assessment**
The flood damage from flooding and the reduction in adverse impacts through implementation of mitigating measures depend mainly on the value to be protected, the inundation process and the proposed/implemented protection level. This can be expressed as:

\[
\forall \ j \ \ B_i^j(P_i^j, P_0^j) = f (\text{inventory of economic value, flooding process, } P_i^j, P_0^j) \\
B_k^j(P_i^j, P_0^j) = f (\text{population, cost of life, flooding process, } P_i^j, P_0^j) \quad (4.4) \\
B_s^j(P_i^j, P_0^j) = f (\text{social disruption, flooding process, } P_i^j, P_0^j) \\
B_a^j(P_i^j, P_0^j) = f (\text{environmental property, flooding process, } P_i^j, P_0^j)
\]

The flooding process includes the failure mechanisms and the assumed pattern of inundation resulting from the discharge of excess water into the polder, and further transfers such inundation levels into damage. The value of the property to be protected as well as the inundation process will be different for each polder.

**Uncertainty assessment**
Considerable uncertainty is associated with estimation of the occurrence probability of flooding situations, the flooding process as well as the evaluation of (generalized) benefits and costs. The different terms in the objective function are subject to the
following uncertainties:

**protection level:**
- inherent uncertainty of the hydro-meteorologic process;
- selection of appropriate extremal distribution and extrapolation of boundary conditions; and
- probability distributions and joint probability of boundary conditions.

**physical flooding process:**
- failure mechanism;
- hydraulic routing in river as well as in polders; and
- inundation process.

**economics, loss of life, social and environmental consequences:**
- inventory of items;
- damage function;
- actual realization (meeting design values and costs); and
- trade-off between objectives (degree of compensation between objectives)

### 4.3.3. Implementation scheduling

Let \( \{ M_j^i, j=1, ..., n \} \) represent a set of measures which can upgrade the protection level to \( P_j^1 \) for polder \( j \). Implementation of \( M_j^i \) may involve a large investment and a large implementation capacity, requiring therefore a phased approach to implementation over a long time period. Implementation scheduling or the selection of the best sequence for implementation of the works can then be formulated as a constrained optimization problem. Several approaches may be taken such as:

- maximization of the present value of the net benefits occurring during the selected planning horizon; and
- minimization of the present value of the (economic) implementation costs over the selected planning horizon for the target protection level. This is a more simple objective but can be justified on the basis that the desirable level of protection has been determined in the previous step. This represents a least cost approach to reach the targets.

Considering the set of the measures for all polders \( \{ M_j^i \} \) and denoting a possible sequence of implementation \( S_n \) by a starting time for each project, namely:
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\[ S_n = (t_1, ..., t_j, ..., t_n) \] (4.5)

then the objective function for the first approach, considering only the economic benefit to simplify the notation, can be formulated as follows:

\[
\max: \forall \text{ possible } S_n \left[ \sum_{j \neq j \neq 1}^{T+t_j} \left( \sum_{i=t_j+1}^{T+t_j} B_j^i(P_i^j, P_0^j) \times \frac{1}{(1+r)^t} - C_i^j(P_i^j, P_0^j) \times \frac{1}{(1+r)^t_j} \right) \right]
\] (4.6)

in which: \( T \) represents the life service of the flood protection system; \( r \) is the discount rate.

The first term in the objective function expresses the present value of the benefits occurring after measure \( \{M_i^j\} \) has been implemented in year \( t_j \), the second represents the present value of the cost of implementing measure \( \{M_i^j\} \) in year \( t_j \).

For the second approach the objective can be written as:

\[
\min: \forall \text{ possible } S_n \left[ \sum_{j \neq j \neq 1}^{T+t_j} C_i^j(P_i^j, P_0^j) \times \frac{1}{(1+r)^t} \right]
\] (4.7)

For practical reasons of implementation planning, the measure \( \{M_i^j\} \) for polder \( j \) may be broken down into a number of sub-measures for dike sections for which scheduling follows the same approach as indicated above. Constraints expressing limits on budgets, implementation capacity and interdependence of projects will further complete the optimization model.

4.3.4. Long term adjustment of the target safety level

Significant changes in the value to be protected, the cost for protection, and changes in hydraulic, hydrologic or even meteorological conditions in the river basin will require to update the target set \( \{P_i^j\} \). Subsequently the implementation of a new set of measures should be determined.

A target set \( \{P_i^j\} \) is determined for the situation at a particular time, e.g. at the end of the planning horizon and will then incorporate as far as possible projected changes in the system. At regular time intervals such projections may be revised followed by an update of the set \( \{P_i^j\} \).
4.4. Solution to the flood alleviation decision problem

The large and complex decision problem requires an approximation and decomposition into manageable sub-problems. A major decomposition was proposed in section 4.3 by splitting the total problem into three parts, namely:
- determination of target protection levels;
- scheduling of the implementation; and
- long term adjustment of protection level targets.

A further decomposition into sub-problems and solution procedure is proposed below which contains the following steps:
1. simulation of the flood volume-exceedance probability relationship;
2. flood damage assessment;
3. uncertainty assessment;
4. optimization of the level of protection, this is carried out in two steps:
   optimization of an overall level of protection, and optimization of the protection level for each individual polder; and
5. optimization of implementation scheduling.

The general approach in the solution of the problem is to follow a "simulation approach" by which the response surface (net benefit) for the flood alleviation problem is traced for a limited and well-chosen set of possible values for the decision variables, namely, alternative sets of \( P^j \) or the associated \( M^j \). Tracing the performance in terms of net benefits and uncertainty range for a selected number of alternative measure sets allows to compare performances and select the optimal set. Step (1), (2) and (3) generate the basic information for decision analysis; step (4) and (5) constitute methods to select the optimal solution using this information.

4.4.1. Simulation of the flooding process and estimation of occurrence probability

Equation 4.3 in the formulation expresses the estimation of the protection level \( P^j \) as a function of the physical configuration, including river schematization and measures, and the boundary conditions. In view of the complexity of this relationship this can only be traced using simulation: given the physical configuration and a particular set of (upstream and downstream) boundary conditions, the amount of water \( W_j \) entering into polder \( j \) can be simulated with a numerical model for water movement in open channels.
Framework and formulation

A particular parameter in this simulation is the failure mechanism; in this study two extremes for this failure mechanism will be explored, i.e. overflowing with breaching and overflowing without breaching. Each failure mechanism will result in different volumes of water entering into the polders. The two cases represent the extremes in terms of water volumes spilled into the polders.

The simulation can be used to characterise the effect for all combinations of the boundaries. Subsequent integration over the contours of equal \( W_j \) for the probability density of the boundary conditions provides the exceedance probability for \( W_j \). This set of simulations should be carried out for different configurations of \( (M_i^j) \).

The use of simulation to generate the information on \( (P_i^j) \) and the relationship with flood water volumes is discussed in Chapter 5.

4.4.2. Flood damage assessment

In the previous step simulation is proposed to determine flood probabilities and associated flood water volumes \( W_j \) for different sets of dike heightening measures \( (M_i^j) \). A further linkage of water volumes with resulting damage is required.

An extensive inventory of damage items is required. Several categories of damages, mainly differentiated according to the nature of the damage (direct or direct) or the measurability of the damage (tangible or intangible) are presented in Fig. 4.9.

Direct damages may be defined as those caused by physical contact with the flood water, while the indirect damages are defined as those that arise from either the physical damage or loss caused by the flood or the very presence of flood. Flood damage which cannot be measured in monetary units is regarded as the intangible damage.

Tangible direct flood damage
This damage component should be assessed based on the land use information for the polders and the physical characteristics of the floods. GIS is proposed to assist the inventory of land use information; a digitized land use map is available in the Netherlands as well as in many other countries.
Fig. 4.9. Categorization of Flood damages

**Tangible indirect flood damage**
Indirect flood damages should be assessed and added to tangible direct damages to form the tangible damage. Few assessment methods are available in the literature. In the present study a method will be developed to roughly estimate the indirect damages of floods covering all impact levels.

**Intangible flood damage**
Intangible flood damages have been reported and assessed mostly in a qualitative way in the literature, and have not yet been successfully incorporated into decision analysis. In the present study the factors which affect intangible damages have been identified as well as a relationship among the various kinds of flood damages. This allows to include intangible damages in the decision analysis.

This component of flood damage assessment is worked out in Chapter 6.

**4.4.3. Uncertainty assessment**

The range of uncertainty for each alternative solution for flood alleviation should be determined so that this information can be incorporated in the decision analysis.

A chain of uncertainties is associated with specification of the total flooding
Framework and formulation

phenomenon. This starts with the inherent uncertainty on the occurrence of flood situations up to the evaluation of damages. A tree of uncertainties is traced for each relevant alternative solution \( \{ M_i \} \). This determines a range of outcomes for the particular set \( \{ M_i \} \). Comparison of alternatives can then be based on the expected value as well as on the size of the uncertainty range.

A further discussion is included in Chapter 7.

4.4.4. Optimization of the protection level

Magnitude of the optimization problem

The number of alternatives and the number of simulations which have to be carried out to generate the necessary information for optimization is very large. Application of a formal mathematical optimization routine to handle the large number of possibilities is not possible due to the complexity (strong non-linearities) of the problem. An example may illustrate the size of the optimization problem: for a modest system of 4 polders for which a set of 10 alternative protection levels are considered, a total of \( 10^4 \) combinations of protection levels have to be examined. Considering further 5 upstream and 5 downstream boundary situations and 2 possible failure mechanisms leads to a total of "\( 10^4 \times 5 \times 5 \times 2 = 500000 \)" simulations.

Solving this optimization problem by brute force enumeration represents a huge amount of work, therefore an approach should be found to cut down considerably on the number of simulations, e.g. by solving subsets in a limited number of successive steps and/or using gradient search techniques. In line with the nature of the problem the following optimization strategy is proposed:

step 1:
Optimization of an average or overall level of protection by considering the same level of protection for each polder. This reduces the number of simulations to "\( 10 \times 5 \times 5 \times 2 = 500 \)".

step 2:
Refinement of the overall level of protection to account for the individual differences of the polders. A gradient search can be used to carry out these refinements with a limited number of extra simulations.
Those steps are elaborated below.

**Optimal overall level of protection** (same protection level for each polder)
The optimization process for this situation is presented in Fig. 4.10. At this stage, the four polders are regarded as a combined system, thus only one protection level is searched for the system, i.e. selection of the same level for each polder results in a total set of only 10 alternatives.

![Diagram](image)

*Fig. 4.10. Selection of an overall protection level*

**Optimal protection level for each individual polder**
The optimization process is presented in Fig. 4.11. At this stage, each polder may have its own protection level, the optimal protection level for each polder may be differentiated from the overall protection level determined at the previous stage. The differentiation can be made by checking the direction in which the overall protection (total net benefit of flood protection) can be improved. With a limited number of steps the overall optimum can be reached.
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Figure 4.11 gives an example where the overall protection level can be further differentiated to two levels upwards in Polder 4, one level upwards in Polder 3, and one level downwards in Polder 1. As illustrated in Fig. 4.7 the response surface is continuous and convex and the search will therefore yield a global optimum.

![Graph showing alternative safety levels vs polder number]

Fig. 4.11. Search process for the optimal protection level for individual polder

4.4.5. Optimization of implementation scheduling

Scheduling of the various components of flood protection can be categorized in the class of sequencing problems for which a wide variety of situations and solution algorithms have been studied.

The scheduling problem selects an optimal schedule out of a vary large number of combinations. A mathematical optimization model may be used to do this optimization. An illustrative formulation which can be solved with a mixed integer programming optimization model is described below.

Objective function:
maximization of the present value of the net benefits:
maximize : \[ \sum_j V_j^l \times X_j^l \] (4.8)

in which:

\begin{itemize}
  \item \( j \): index of a particular flood control project (measure);
  \item \( l \): starting year index;
  \item \( V_j^l \): present value of the net benefit of project \( j \) when it starts in year \( l \);
  \item \( X_j^l \): 0 or 1 integer variable, if \( = 1 \), then project \( j \) starts in year \( t \).
\end{itemize}

**Constraints:**

(1) a particular project can only be selected once into the schedule or not at all:

\[ \forall_j \sum_i X_j^i \leq 1 \] (4.9)

(2) the requirements for implementation capacity of all flood control projects in progress at a certain time \( y \) should not exceed the maximum available implementation capacity:

\[ \forall_y \sum_j \sum_l I_j(l, y) \times X_j^l \leq A^y \] (4.10)

in which:

\begin{itemize}
  \item \( y \): construction year index;
  \item \( I_j(l, y) \): use of implementation capacity by flood control project \( j \) in year \( y \) after it started in year \( l \);
  \item \( A^y \): maximum limit on available implementation capacity in year \( y \), expressed e.g. by the maximum length of dike upgrading that can be realized in a particular year.
\end{itemize}

(3) the budget requirements for all flood control projects in progress at a certain time \( y \) should not exceed the maximum available budget:

\[ \forall_y \sum_j \sum_l B_j(l, y) \times X_j^l \leq E^y \] (4.11)
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in which:
  \( B_j(l, y) \): use of budget in year \( y \) by flood control project \( j \) after it started in year \( l \);
  \( B^\text{max}(y) \): maximum limit on available budget in year \( y \).

Many alternative formulations for the scheduling problem can be considered to reflect particular circumstances during implementation.

4.5. Conclusions

A framework of the dynamic multiobjective decision model under uncertainty for dike heightening has been presented in this chapter. Due to the different objectives of dike heightening and due to the different measuring units of different aspects of flood damages, and the cost and the adverse impact of dike heightening, a multiobjective decision model is required. Due to the uncertainties in estimating the values of the variables in the model, the decision can only be made in a multiobjective framework under uncertainty. When dike heightening is a substantial project or program, in terms of the cost and the spatial coverage, phasing of dike heightening should be arranged. Different phasing of dike heightening influences the expected flood damage in the time period after the completion of dike heightening at any section. Therefore, the variables in the model change with time, which requires a dynamic approach.

The framework and the formulation presented in this chapter show that it is not easy to search an economically optimal protection level, partly due to the hydraulic interdependence of dike heightening. The effectiveness in reducing flood damage can be increased if structural and non-structural measures are combined. However, incorporating any measures of flood alleviation which will influence the hydraulic situations around polders would increase the difficulty in solving the optimization problem.

For dike heightening in the river transition zone, full differentiation of protection levels and full optimization of the dike heightening phasing involve extensive calculations. Therefore, simplifications are necessary. In this chapter, three rounds of optimization are suggested to approach an operational risk assessment in flood alleviation focusing on dike heightening.

In Chapter 8, an application will be worked out to determine the economically optimal
dike heightening in the river transition zone in the Netherlands. In this application, the optimal protection level will be sought and the differentiation from the optimal level over polders will be indicated. Optimization of implementation scheduling will not be included in the application.

This chapter sets the stage for the subsequent chapters of Part III.
Part III

Operationalization
CHAPTER 5

FLOOD MAGNITUDE
AND OCCURRENCE PROBABILITY
IN THE RIVER TRANSITION ZONE

5.1. Introduction

Given hydrologic conditions at boundaries, the unsteady overflow discharge can be calculated through hydraulic simulation, and its occurrence probability can be estimated by transferring the probability distributions at boundaries to river segments in the potentially flooded area. Flood routing simulation (calculation) in the channels and on flood plains has been of interest to hydraulic engineering for well over a century. There are basically two governing equations of unsteady flow—continuity and motion—which are two partial differential equations expressing the physical law of conservation of mass and conservation of momentum. Detailed derivation of the equations of continuity and motion can be found in many reference works such as Van te Chow (1959), Liggett (1975) and National Environmental Research Council (1975). The numerical solution of these equations has been implemented in many models and associated computer programs.

In addition to the flood routing in channels and in flood plains, the flood routing in polders can also be considered. The unsteady overflow (and breach) discharge at dike sections determines the spatial and temporal process of flood routing in polders.

The intent of this chapter is not to improve the techniques or methods to solve the governing equations of unsteady flow. Instead, it is, as indicated in Chapter 4, to formulate methods and procedures for the estimation of flood water volume entering polders in the river transition zone and to determine the exceedance probability. This involves calculation of unsteady dike overflow discharges from the potential flood events with a one-dimensional open channel flow model.

The actual process of overflow in combination with the breaching of dikes has received increasing attention in surveys of the causes of dam failure (e.g. AlQaser and Ruff, 1993; Dewey and Gillette, 1993; Frawley and Von Thun, 1993) and experimental
research (e.g. Bell et al., 1992; Steetzel and Visser, 1993; Visser, 1994). The type of
dike material, dike slope, surface material and flow process all have influence. The
duration of the high water level condition in front of the dike further influences the
stability of the dike, the eventual failure and the ultimate inundation level in the polder.

Two extremes are considered for the overflow failure mechanism: overflow without
breaching, and overflow with breaching to the extent that the polder is ultimately
inundated up to the lowest dike height surrounding the polder.

In this chapter, after an introduction to the river transition zone, the state of the
hydrologic variables in the river transition zone will be examined. A procedure will be
presented to simulate the dike overflow discharge. Finally, a method will be introduced
to estimate the exceedance probability of the overflow volume. The difference between
the extreme failure mechanisms will be illustrated.

5.2. River transition zone

In a catchment area with a sea boundary, at least two boundary conditions are generally
needed for river flow calculation: the sea level as a downstream boundary, and river
discharge as an upstream boundary.

As mentioned in Chapter 1, the river transition zone is that part of the catchment area
where the upstream and the downstream boundary interact. To identify the transition
zone in a catchment, one should examine the relationship of hydrologic variables in the
particular zone with the boundaries. The following conceptual set of curves, in which
the local water level (representing the local hydrologic variable) is set out in function
of boundary conditions, can help to identify the transition zone. In Fig. 5.1, $Q_u$ and $H_d$
represent the upstream discharge and downstream sea level, respectively.

The transition zone is presented in the figure ranging from a flow condition which is
purely dominated by the upstream discharge towards a condition which is completely
dominated by the downstream sea water level. The transition zone can be defined as that
zone in which the two boundaries have to be considered in order to analyze the local
water level situation. In the figure, three intermediate stages are presented, namely:

- an upper transition zone where the river discharge still dominates and the sea level
  boundary has minor influence;
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- a middle zone where both boundaries are of comparable influence; and
- a lower part where the sea water level dominates.

Fig. 5.1. Contour lines of local water level at different zones of a catchment area

To assess the exceedance probability of the local hydrologic variables, one needs to link the hydrologic variables of the boundaries, which have estimated exceedance probabilities under the present natural conditions, with the hydrologic variables at the sites of interest. Obviously, when the natural conditions related to upstream river discharges and the sea level change in the long run, then the exceedance probability at boundaries should be re-estimated accordingly.

5.3. Steady and unsteady boundary conditions

A boundary condition including up- and downstream boundary conditions and a river schematization, representing information about the river network and the cross-sections of the river, are important input data of a river flow model.
A distinction can be made between steady and unsteady boundary conditions. The type of flow condition in the transition zone depends on the selected type for each of the boundary conditions. The possibilities are presented in Table 5.1.

Table 5.1. The state of hydrologic variables in the transition zone

<table>
<thead>
<tr>
<th></th>
<th>steady state of $H_d$</th>
<th>unsteady state of $H_d$</th>
</tr>
</thead>
<tbody>
<tr>
<td>steady state of $Q_u$</td>
<td>steady state</td>
<td>semi-unsteady state</td>
</tr>
<tr>
<td>unsteady state of $Q_u$</td>
<td>semi-unsteady state</td>
<td>unsteady state</td>
</tr>
</tbody>
</table>

The flow condition in the transition zone resulting from two steady boundary conditions can be termed steady, while the combination of one of the boundary conditions as steady and the other as unsteady can be called semi-unsteady.

In reality both boundary conditions are unsteady and the challenge is to account for the timing of the two critical peak conditions and the joint probability of occurrence. The occurrence of the individual boundary conditions is usually expressed as the annual occurrence (exceedance) frequency, expressing the probability that the event occurs in any given year. In addition to the annual occurrence frequency, the joint occurrence of the two events requires further to consider the timing of the events during the year. Obviously the effect is much different if the two events occur at the same time or at a different time during the year.

To simplify the hydraulic calculation in practice, the peak value of the river discharge wave and/or tidal peak sea level have been used as steady boundary conditions. For example, one of the boundary conditions has been considered steady and kept constant at the peak level in the hydraulic calculation in the Dutch part of the Rhine by the Dutch water authorities (Rijkswaterstaat). A hydraulic calculation at both steady and semi-unsteady state obviously presents an overestimation of the flooding situation. In the present study, both boundary conditions will be taken as unsteady but assumptions will have to be made about the timing of the two boundary events.

There is usually relatively little information about the joint occurrence (timing) of the two boundary events. For example, the only information on the joint occurrence of the discharge at Lobith and the sea level at Hoek and Holland that could be found is given by Van der Made (1969) who concluded from a statistical analysis of river flows during storm surge (sample of 28 events) that there appears to be no correlation between storm
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surges at sea and large river discharges.

The conceptual shapes of the boundary conditions are presented in Fig. 5.2 and 5.4. The shape of the so-called design discharge wave of a certain peak discharge is presented in Fig. 5.2. A high peak sea level usually results from the normal tidal movement and a wind set-up (Fig. 5.3), the combination of which creates the so-called design sea level surge (Fig. 5.4). The duration of the upstream design discharge is much longer than that of the design sea level surge.

![Discharge vs Day Graph](image)

Fig. 5.2. Design discharge wave at the upstream boundary

It can be readily inspected from the physical characteristics of the flooding pattern that the phasing of the two boundary conditions with respect to each other will have a strong influence on the resulting river water level and dike overflow discharge at particular dike sections. It can further be expected that this phasing will be more important in the middle section of the transition zone than in the upper and lower section where one of the boundary conditions is the more dominate one.

5.4. Procedure for simulation of dike overflow

5.4.1. Combinations of boundary conditions

A wide range of situations corresponding to all possible occurrence frequencies has to
be analyzed for flood alleviation for the transition zone. A systematic analysis of all these situations can best be structured according to combinations of the boundary conditions. Each boundary condition can be characterised by a peak value. For example, consideration of a range of 10 peak values for the sea level and 10 values for the peak discharge, implies that a total of 100 boundary combinations and associated potential flood situation are to be investigated.

Fig. 5.3. Tidal movement and wind set-up

Fig. 5.4. Design sea level surge

The probability density functions of the boundary conditions, determined on the basis of a ranked series of maxima, indicate the probability that a particular (extreme) situation occurs during any particular year.
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As mentioned in Section 5.3 the potential flood situations arising from a particular combination of (annual) boundary conditions, should further be characterised by the time phasing (within the year) of the two boundary conditions.

Ideally the effects of all possible different phasing situations should be evaluated and their probability of occurrence determined. The expected value of the different possibilities can then be considered as the flooding consequence associated with the particular combination of boundary conditions.

The effects of different phasing can be determined by simulation of the different situations. However relatively little information is available on the probability of occurrence of each phased situation. Different situations ranging from complete independence to full dependence could be considered for the two events. A completely random occurrence of the two boundary conditions will result in a minimum flooding consequences and a maximum can be expected if the two events are fully dependent on each other and coincide.

It was felt that there is at this stage no sufficient information to decide on any one of the range of possibilities. Further research is required and should provide more information and an appropriate approach. It was therefore decided to use the maximum interaction in this study as it remains closest to the current practice.

The effects of different phased situations have been simulated and the situation with the maximum effect was selected. This represents obviously an overestimation of the expected value. It represents however a downward adjustment of the estimate in current practice (semi-unsteady condition). It can be expected that additional research will yield a further downward adjustment.

The next sections summarize the simulation procedure for dike overflowing incorporating the above outlined approach for a combination of the boundary conditions.

5.4.2. Dike overflow followed by breaching

In the extreme case of overflow with breaching, the ultimate inundation level in a polder can be assumed to equate the lowest dike height of the polder. Because of this assumption, no hydraulic calculation is needed for the inundation level in the polder. Nevertheless, it is necessary to investigate two aspects relevant to the characteristics of
5.4.3. Dike overflow but no breaching

River schematization implementation
If the overflow discharge is regarded as a lateral outflow from the river, then a common river flow model can also be used to calculate the overflow discharge in case that the input data describing the river schematization are adapted according to the relevant dike heightening alternative.

If the river model does not have a function of overflow simulation, such a function can be added to it: some "artificial" river sections and a sluice can be appended in the river schematization to the dike sections where overflow is considered; the bottom level of the sluice is then taken at the dike height.

Dike overflowing simulation procedure
Following the approach as outlined above, the dike overflow simulation procedure can be summarized into five steps (Zhou and Van der Heijden, 1994-b) (see Fig. 5.5):

1. Set a magnitude range of the peak value for $H_i$ and $Q_a$, and form a set of discrete peak values within the range for each of the boundary conditions.

2. Take a certain peak value of the upstream discharge and a certain peak value of the downstream sea level out of the set to form a boundary condition combination.

3. Construct, respectively, the unsteady boundary conditions according to the design discharge wave and the design sea level surge. The timing of the peak values at up- and downstream boundaries which results in the maximum interaction in the transition zone should be selected.

4. Calculate the overflow discharge $Q_i(t)$ at the dike section $i$ with the river flow model for each boundary condition combination. The overflow volume at the dike section can be obtained by integration of $Q_i(t)$ (Equation 5.1), and the total overflow volume at all dike sections of a polder $j$ can be obtained by summation (Equation 5.2.)

5. Take another boundary condition combination: repeat the step (3), (4). The
possible maximum overflow volume under another boundary condition combination can now be obtained.

\[ W_i = \int_0^\infty Q_i(t) \, dt \]  

\[ W_j = \sum_i W_i, \quad \text{where: } i < j \]  

Fig. 5.5. Dike overflowing simulation procedure

(6) Repeat step (2), (3) and (4) until the range of peak values of the boundary conditions is covered. After this step, a relationship between the boundary condition combination and the possible maximum overflow volume in polders can be established.
5.5. Estimation of the exceedance probability of overflow volume

With the relationship between the boundary condition combination and the water volume entering a polder, the exceedance probability of the water volume in the polder can be estimated. The following approach is adapted for such a purpose from Van der Made's method (1969), with which he estimated the exceedance probability of the water level in the river transition zone in the Netherlands. This study adapts this approach to estimate the exceedance probability of the overflow volume in a polder in the transition zone for two extreme dike failure mechanisms.

5.5.1. Establishing contour lines of overflow volume

Through the above simulation procedure, the possible maximum overflow volume in polder j can be calculated, given the dike heights of a polder, for all boundary condition combinations.

The possible maximum overflow volume in the polder "j" \( W_j \) and the corresponding peak values of \( Q_u \) and \( H_d \) at boundaries can be plotted in a coordinate system. A certain value of \( W_j \) can be the result of different boundary condition combinations. Therefore, contour curves of equal \( W_j \) in function of \( Q_u \) and \( H_d \) should be drawn. Figure 5.6 illustrates the contour curves for the overflow without breaching situation.

5.5.2. Estimating the exceedance probability of overflow volume

The exceedance probability of \( W_j \) can be estimated on the basis of the probability distributions of the hydrologic variables at both boundaries.

If the exceedance probability distributions of \( Q_u \) and \( H_d \) are independent from each other, then the exceedance probability of a particular value of \( W_j \) is determined by integration of the joint probability density distribution of \( H_d \) and \( Q_u \) over the area outside the \( W_j \) contour line (Fig. 5.7). This can be expressed as:

\[
P(W_j) = \int_{\Omega} \int p(Q_u) \times p(H_d) \, dQ_u \, dH_d
\]  
(5.3)
**Flood magnitude and occurrence frequency**

in which: $\Omega$ is the complement area of $(1-\Omega)$, which is the shadow area in Fig. 5.7 enclosed by the contour line of a particular value of $W_j$, defined over the $H_d$- and $Q_u$-axis; $p(Q_u)$ and $p(H_d)$ are, respectively, the probability density functions of $Q_u$ and $H_d$.

![Contour lines of $W_j$ as a function of $Q_u$ and $H_d$ in the transition zone](image)

**Fig. 5.6.** Contour lines of $W_j$ as a function of $Q_u$ and $H_d$ in the transition zone

![Integration for the exceedance probability of a particular $W_j$](image)

**Fig. 5.7.** Integration for the exceedance probability of a particular $W_j$

The exceedance probabilities of other values of $W_j$ can be obtained similarly. From those, an exceedance probability curve of $W_j$ can be established.

The hydrologic variables of the potential flood events differ with different dike
heightening alternatives. When an alternative of dike heightening is implemented, then
the river schematization changes accordingly. Consequently, the overflow volume \( W_j \) as
a function of \( Q_e \) and \( H_d \) and the exceedance probability curves of \( W_j \) should be
established for each dike heightening alternative.

For each flood volume \( W_j \), the inundation level and inundation depths in the polder \( j \) can
be derived on the basis of the topographic characteristics of the polder.

5.6. Conclusions

The procedure and method put forward in this chapter aim to properly describe the
overflow discharge of potential flood events using a river flow model. With different
simulation procedures, the same river flow model can reach results with different
accuracy. The procedure presented in this chapter can be used to calculate the overflow
discharge for the extreme of overflow without breaching. With the calculated overflow
discharges under various boundary conditions, the inundation depths in polders and their
exceedance probabilities can be estimated. Determination of the ultimate inundation level
in polders for the extreme of overflow with breaching is easier: one can assume that the
inundation levels in polders ultimately equal the lowest dike height of the polder if river
water level exceeds the dike height. The method to determine the exceedance probability
of the water volume in polders is applicable to both extremes of dike failure.

The peak values of the sea level and the upstream discharge show strong cyclical and
seasonal fluctuations. Ideally the effects on flow conditions of all possible different
phasing of the peak values at boundaries should be evaluated and their probability of
occurrence estimated based on the information on the joint probability of the different
phasing. Considering the lack of sufficient information, it was decided to calculate the
water level and overflow volume resulting from the maximum interaction of the
boundary conditions in this study as it is the method closest to the current practice.

The procedure prepared in this chapter is applicable to a catchment area with one
downstream and one upstream boundary. However, in some situations, more than one
river tributaries join with each other before running into the sea (Fig. 5.8). In that case,
it would be difficult in practice to capture the linkage between hydrologic variables at
three gauging sites (boundaries) and locations of interest in between. At first, if each
boundary represents a dimension, the relationship is three or multi-dimensional, and a
mathematical expression of the relationship is therefore probably difficult to formulate. Furthermore, since the exceedance probability of flood should be calculated by integrating the joint probability functions of all boundary conditions over the multi-dimensional space, it is also likely to be inherently difficult.

Fig. 5.8. Multiple boundaries of a catchment

Nowadays, the computer program of a river flow model is usually used for hydraulic calculations. Most river flow programs can be executed only under one boundary condition combination. Therefore, applying this procedure will certainly cost more time. Technically speaking, however, it is not difficult to write a computer program, based on an executable river flow program to simulate the dike overflowing under more boundary condition combinations. In that case, the extra time needed is very affordable. Consequently, the procedure and method for flood routing simulation are applicable.

The simulation procedure and method for the determination of the probability of the water volume entering in polders developed in this chapter will be applied in Chapter 8 to study the dike heightening in the river transition zone in the Netherlands.

In the next chapter flood damage assessment, another important component in the decision analysis for dike heightening, will be discussed.
CHAPTER 6

FLOOD DAMAGE ASSESSMENT

6.1. Introduction

The physical process of flooding was described in the previous chapter. The purpose of this chapter is to assess the various kinds of damages associated with the flooding process.

The damage caused by flooding evidently differs with the magnitude of the flood and the characteristics of the flooded area. According to the magnitude of the flood damage, floods have been classified as follows (Avakyan and Polyushkin, 1991-b):

- small flood (basically only field/rural areas are flooded);
- medium flood (in addition to fields, arable land and some rural settlements are flooded);
- large flood (cities to some degree and main railways and highways are flooded); and
- catastrophic flood (flooding of an important area with great damages to cities, main highways and railways, to flood-resistant buildings and public facilities, and even to human life).

The assessment methods presently available are either at a fine geographic level—for example, the unit-loss method—or only suitable for the assessment of a severe flood event—for example, regional economic modelling. Accordingly, assessment methods are required for a flood alleviation study at the regional level or the level of a river basin, as well as for less severe flood events.

Generally, flood damages include human life loss and social, economic, environmental and other effects (see Fig. 1.5). Flood damages can be categorized, as described in Chapter 4:

- according to the nature of the damage: direct or indirect effects; and
- according to the measurability of the damage: tangible or intangible effects.

Tangible direct flood damages may be defined as those caused by physical contact with
the flood water, whereas indirect flood damages are defined as those that arise from either the physical damage or loss caused by the flood or the very presence of flood. Tangible damages are restricted to those that can be quantified and expressed in monetary terms, thereby treating qualitative damages and quantitative damages that cannot be expressed in monetary values, as intangible (see Fig. 4.9). The human life loss issue is usually addressed separately.

With these categories, four types of flood damages can be formed: the tangible direct, tangible indirect, intangible direct and intangible indirect damage. For the last two kinds usually a combined assessment is made (e.g. Green and Penning-Rouses, 1986).

In this chapter, based on a literature survey of a previous study (Zhou and Van der Heijden, 1994-c), definitions and content of various kinds of flood damages will be introduced. Assessment methods for flood damage will be reviewed and, if needed, developed. The magnitude relationship of different kinds of flood damages will be explored. With such relationship(s), some kinds of flood damages, which are usually difficult to capture, may be estimated on the basis of knowledge of the other kinds of damage.

6.2. Tangible direct damage

6.2.1. Definition and content

Tangible direct flood damages may be defined as those caused by physical contact with the flood water. They include (e.g. Kates, 1965; Baecher, et al., 1980):
- damage to or loss of buildings and their contents, including furnishing, equipment, decorations, stocks of raw materials, materials in process and completed products;
- damage to or loss of infrastructures (road, railway, bridge, tunnel, etc.) and public utilities (e.g. sewage treatment, gas, electricity, water supply, telecommunication);
- damage to or loss of vehicles and boats;
- loss of livestock; and
- damage to or loss of environmental objects.

The direct flood damage is strongly related to the accumulated properties located in the
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potential flooded area. Various features may result in differences in flood damages: the vertical distribution of the contents of the particular property type, types of goods and building fabric and consequently the susceptibility, and the economic value of individual goods. The susceptibility can be defined as the probability and the extent to which the physical presence of water will affect inputs or outputs of an activity. Therefore, the tangible direct flood damage is larger in an urbanized area than in a rural area, respectively, in industrialized countries than in developing countries.

6.2.2. Assessment method

The unit-loss method is based on the investigation of damages to individual land use items, generally categorized in terms of physical structures (e.g. bridges), buildings (including dwellings) and properties (e.g. agricultural land) (Parker, 1992). The basic task therefore is to identify all relevant units and, in addition, to identify their values. Although the method is basically meant to estimate stock losses, the unit could also be defined in terms of flow losses. The problem is, of course, the identification of such units. The unit-loss method has been applied, according to for example Parker (1992), to assess the direct flood and earthquake losses, both potential and actual.

There are three ways of calculating the loss values:

- future service value: the present value of projected annual services;
- replacement value: the present value of the cost replacing the property providing essentially same service over the same lifetime; and
- restoration value: the present value of the cost of repairing and restoring the property to its pre-flood condition.

In the absence of reliable data on the value of services provided, pre-flood market prices may be used.

In applying the unit-loss method, the first step is to inventory the relevant unit. Das and Lee (1988) in this context made a distinction between the so-called traditional and nontraditional approach. The traditional approach uses only the data of direct field survey and investigation. The nontraditional approach uses data compiled from available or secondary sources such as—in the USA—the U.S. Census data tapes, Robert Morris & Association data, County Business Patterns and others. In the Netherlands, the Central Bureau of Statistics (CBS) can generally provide information of this kind. Field survey and data inventories on residential, commercial, industrial and public structures are used
selectively only for data validation purposes or for the identification of special or unique structures. This approach is carried out at a geographic scale for which the economic data are readily available from these secondary sources. The spatial units are city blocks (for highly urbanized areas), enumeration districts and census tracts. Spatial units of the smallest size are preferably used so as to retain as much spatial detail as possible. Commercial and industrial data of individual establishments at specific addresses are assigned to the corresponding spatial units. Agriculture and public sector economic data are compiled at the grid level where maps constitute the main sources of data.

The second step is to value the inventoried units. The flood loss estimation includes the following steps: (1) collect data on the values of properties through field surveys and/or interviews; (2) develop depth-loss functions for the study area from generalized functions, post-flood loss surveys, statistics, or hypothetical depth-loss estimations to provide estimates of the loss to structures, buildings and properties as a function of the depth of flooding; and (3) calculate the value per unit and aggregate the losses.

There are two basic limitations to the unit-loss modelling methods. The first is the requirement of collecting and updating all the information about the properties (e.g. pieces of land, buildings) and their values. Secondly, the method is limited by its capability to handle damages of a secondary or higher order.

As mentioned above, tangible direct flood damage assessment needs data of two aspects: the land use information in the floodplain and the flood loss values to individual properties. The unit-loss method is widely accepted as a method for calculating tangible direct losses. One of the main problems with this method is to collect the right data to reach a valid level of assessment. In many countries, this might be very difficult. In some countries, however, the registration of spatial objects and functions by digitized land use maps is at a very reasonable level. This spatial data can be used as property inventory in the unit-loss method to assess the potential flood losses.

6.2.3. GIS-assisted land use inventory

Efforts to integrate water related problems into a GIS have been located in the literature. Recent publications covered the fields of hydrologic modelling (e.g. Muzik and Chang, 1993; Warwick and Haness, 1994), floodplain analysis (e.g. Greenwood, et al., 1994), watershed analysis (e.g. DePinto, et al., 1994), water resources management (e.g. Kilgore, et al., 1994), construction site selection (e.g. Olouta, et al., 1994), water quality
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models (e.g. Tsihrintzis, et al., 1994; Cheng, et al., 1994), and assessment of cost, and the social and ecological impact due to water transfer projects (e.g. Stansbury, et al., 1991).

Most GIS's have a limited capability of integrating other programs. The programming capability of a GIS is generally used to export/exchange the GIS database to a computer program written in another language, and the computer program is therefore called "GIS-based" or "GIS-assisted" (e.g. Lieste, et al., 1993).

In the Netherlands, a lot of effort has been put in the registration of all spatial data in the form of digitized land use maps. This offers the possibility to apply some modern computer tools such as GIS to inventory the land use information, thus to assist the assessment of tangible direct flood damages. In addition, applications of GIS for property inventory in the floodplain open a door to integrate non-structural measures such as the land use zoning to structural flood alleviation measures.

In this study, the recently developed digitized land use map will be applied for the property inventory in Dutch polders (Zhou et al., 1995-b). The digitized land use map will be manipulated by ARC/INFO GIS. Among a few hundred GIS software programmes, ARC/INFO is widely considered to be one of the most sophisticated.

6.2.4. Flood loss value information

It is always necessary to develop standard direct flood loss functions of various properties for countries in which flooding is one of the major natural disasters, because these functions are essential for flood damage assessment and flood alleviation study. In developing such standard loss values, data in other countries can be referred to, or sometimes, can be transferred directly if the causative factors are similar.

The direct flood loss values to properties vary evidently among countries and with the development stage of a country. The real direct losses to the properties, however, are dependent on the characteristics of those properties, for example, the value of an individual building in a developing country is not necessarily lower than that in a developed country.

In this study, to develop standard depth-loss functions for various properties, a search of the English and Dutch language literature has been conducted to identify the
assessment methods and data of flood losses.

The following are some important developments in the context of tangible direct flood damages based on the literature survey (Zhou and Van der Heijden, 1994-c).

In the United States of America

Friedman and Roy (1966), as referred by White and Haas (1975), constructed, in the United States during development of the National Flood Insurance Program, a computer simulation model for inland (riverine) flooding to estimate the flood frequency and flood depth and to assess the expected damage with flood loss functions.

In 1970, the Federal Insurance Administration published data relating the depth of flooding to the estimated loss for dwelling types. In 1974 these data were revised (Johnson, 1985). These loss data were expressed as a percentage of the total value of the dwelling and furniture at different flood depths. The 1974 depth-loss data were developed using flood insurance claims. The criteria used in collecting these data were: (1) the value of residential structure and content was based on present replacement cost; (2) the flood loss to residential structure and content was based on replacement or repair cost.

In 1979, as a part of a flood control study in the Wyoming Valley area of Pennsylvania, the furniture values of 235 houses were investigated (Appelbaum, 1985). The furniture and appliances in each house were inventoried and the surface area of houses was divided into four size classes. To represent differences in value of average furnishings due to income differences and other factors, these furnishings were rated at three price levels (low, average, high).

Depth-loss relationships for public utilities, transportation, and emergency care were developed from data on losses experienced in the Wyoming Valley due to the Tropical Storm Agnes flood of 1972. This flood caused depths of flooding up to 3-6 m. The study claimed that the average number of cars per household was 1.5; a car would be flooded by a chance of 10%, and the average loss was $2,413. Although the loss to public utilities, roads and cars, and emergency care cost in that study may not be applicable elsewhere, it is still mentioned here since there is very limited similar data. The flood damage to commercial and industrial business was investigated in that study as well (but loss values are not available in the article of Appelbaum).
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In the United Kingdom

From the early 1970's on, the Flood Hazard Research Centre (FHRC) at the Middlesex University, has investigated the benefits of flood alleviation and land drainage. In 1977 a manual was published (Penning-Rowsell and Chatterton, 1977) which sought to systematize the assessment of the benefits of flood alleviation for both urban areas and agricultural land. The manual became a standard reference work on the economics of flood alleviation. Agricultural benefit assessment is the only area where that research has now been shown to be somewhat flawed. The urban flood alleviation benefit assessment data and procedures established in that manual have been successfully used so far without fundamental criticism.

Ten years later, another manual was published (Parker, et al., 1987). Unlike the previous manual, this one only contained data on urban flooding. One of the most important progresses made in this manual was its inclusion of some indirect flood losses.

During the visit to FHRC, the author learned that some flood loss values developed by FHRC (N’Jai, et al., 1990) have been transferred to other countries, e.g. Australia, Portugal and Germany.

In the Netherlands

Since the flood disaster in the Netherlands in 1953, a series of civil engineering works has been carried out for flood protection. In that context, some flood damages were assessed.

The first somewhat systematic research was carried out by Duisier (1984). The research was based on the estimation by various insurance companies, and data from statistical bureaus and governmental departments. Since around the beginning of the 1990's, various research was carried out to study the acceptable level of flood risk for polders under hypothetical dike breaking scenarios and flood loss values of some properties were presented (Vrouwenvelder and Wubs, 1989, 1992-a and 1992-b).

In other countries

The sources found generally fall into four categories: flood-loss studies; dam-failure studies (including flooding routing simulation); descriptions of flood events; economic evaluation of life loss. As far as the flood loss assessment is concerned, besides the works mentioned above, a reference to an unpublished Ph.D. dissertation in Australia (Higgins, 1981) was found, in which the methods and data of flood loss were presented.
Additional studies were conducted by the Centre for Resources and Environmental Studies at the Australian National University.

The completeness, applicability and transferability of the flood loss values requires careful consideration. Data presented in N’Jai, et al. (1990) and in Vrouwenvelder and Wubs (1992-a) have been directly used to make standard depth-loss functions of various types of property for further application in this study (Zhou and Van der Heijden, 1994-c).

6.3. Tangible indirect damage

6.3.1. Definition and content

In contrast with direct damages, indirect flood damages are defined as those that arise from either the physical damage or loss caused by the flood or the very presence of flood. For example, the production of a factory can be stopped as a result of the damage to or loss of the production equipment, or due to the siegement by flood water. Indirect damages include (e.g. Penning-Rowsell and Parker, 1987):

- the production or service losses due to the disruption of network infrastructure and the public utilities; and
- the cost of the emergency measures and cleanup.

According to the Corps of Engineers (Baecher, et al., 1980), emergency costs include additional expenses resulting from a flood that would not otherwise be incurred, such as evacuation, reoccupation, flood fighting, disaster relief, increased expenses of operations during the flood, increased costs of police, fire or military patrol, and abnormal depreciation. Emergency costs should be determined by a specific survey or research and may not be estimated by applying fixed percentages to the physical loss estimations.

The magnitude of the indirect damage can be expressed in terms of the vulnerability, which has three causal factors: transferability, dependency and susceptibility (Parker, et al., 1987). The transferability is the ability of an activity to respond to a disruptive threat either by deferring or using substitutes or relocating. The dependency is the degree to which an activity requires a particular good as an input to function normally. The susceptibility is the probability and the extent to which the physical presence of water
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will affect inputs or outputs of an activity.

The magnitude of indirect damage due to a flood reflects the difficulty of adjusting the economy to compensate for resource losses and production capacity. In general, the greater the indirect damage and the more specialised the affected economic function, the greater the effect upon the total economy will be.

The difficulty to bring the effected functions back into full operation after the flood is an effective indicator of the success of the economy’s adaptive power after flood. The indirect damage is low when the economy can easily adapt to flood, because it has surplus resources, and high when resources affected are difficult to replace. Similarly, the indirect damage to a firm is low when it can readily and rapidly recover from a flood, for example by working overtime. Conversely, the damage will be high when production cannot be recovered rapidly and part of the market will be temporally or permanently lost to competing firms.

If an economic sector inside or outside the flooded area is not able to supply to or receive goods or services from economic sectors directly hurt, this first sector suffers a secondary economic damage. The secondary economic damage occurs only if there is no possible substitution, for example, when the possible sources of supply inside the nation already operate at their full capacity.

A national or regional input-output table can give an insight in the inter-sectoral relationships and support an estimate of the secondary economic loss in conditions of no-substitution. Measuring indirect flood damages is complicated by the difficulty of determining the multiple effects, the problem of separating real damages from (temporal) shifts within the economy or within the time scale, and the need to account only for marginal expenditures due to a flood event rather than the total post-flooded recovery costs (Parker, 1992).

Thus, indirect economic damages for flooded retail properties are effectively zero because shoppers can usually buy equivalent goods from another shop without incurring any significant additional costs. The same is generally true for warehouses since again there are usually alternative sources of supply. It is broadly true for other service activities as well.

Evidently, the indirect (financial) damage a firm suffers and the economic damage to
the nation can differ widely. If the loss of production by one firm is rapidly and cheaply compensated by other competing firms, then the indirect damage to the nation is only the marginal additional cost of production by the second firm, while the financial consequence to the flooded firm may be substantial.

6.3.2. Assessment methods

Regional economic modelling is the only assessment method for indirect flood damage found in the literature. This method is only applicable for severe floods. Assessment method suitable for both severe and less severe floods are required in flood alleviation. The value added method is developed in this study for this purpose.

Regional economic modelling
The typical assessment method for indirect economic damages of a disaster is the use of regional economic models based upon an analysis of the interdependency among the various economic sectors of the regional economy. These models attempt to model the 'ripple' effects of extreme events on regional economies. The analysis focuses on quantifying the damage of an extreme event such as a flood or an earthquake on the output of these sectors (Parker, 1992).

With the exception of research undertaken in Australia by Higgins and Robinson (1982), such models have hardly been used to assess flood damages (Parker, 1992). However, regional economic modelling is now being extensively used to evaluate the damages of large earthquakes, in particular to estimate the indirect damages which were previously difficult to quantify.

The application of the regional economic modelling approach for the evaluation of indirect damages of disasters has been clearly illustrated in for example Kawashima and Kanoh (1990), and Report 2 (1989), while the method for evaluating the decrease of productivity in individual economic sectors, e.g. the employment impact and retail sales impacts, is addressed in a study by the Association of Bay Area Governments (1991).

Regional economic modelling methods use the flow concept of loss, i.e. try to track the transferring process of the flood damage through these relationships among the industrial or economic sectors. They utilize input-output models (tables) in which the input-output relationships of various major industrial sector of the economy are defined within a matrix model. The indirect effects of an extreme event on the entire regional economy

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may then be modelled by adjusting the output of effected sectors caused by for example loss of materials and goods, disruption of production facilities and disruption of transportation facilities.

One point that should be noted in the application of this modelling approach is that the technical coefficients within the input-output table are generally to be assumed constant during the years considered in the analysis. Although the disturbance of economic activities would be gradually recovered over time after the disaster, the process of recovery is difficult to be considered in this analysis.

The basic advantage of regional economic modelling is that potentially the secondary and high order consequences can be estimated. This has proved to be difficult with the unit-loss method. The disadvantage lies in the assumptions on the nature of the quantitative relationships between the various sectors. They are very difficult to deduce from regional economic statistics. Moreover, these statistics do not account for all relevant relationships and therefore represent a strong reduction of reality. For detailed analysis, the unit-loss method has clear advantages.

Value Added method

To estimate the expected (annual) flood damage for flood alleviation not only severe flood events but also flooding with a medium severity should be taken into consideration. Generally, the damages of those medium flood events are not so substantial that they can be measured by using the regional economic models. The "value added" method has been developed by the author to roughly evaluate the tangible indirect damage associated with the productive and service disruption (Zhou and Van der Heijden, 1994-d).

Although some efforts have been made in the UK (Parker, et al., 1987) to link the surface area of the site (or building) and the activities of a plant (e.g. an industrial factory and a hospital) with its indirect flood loss value, the information is hardly transferable to other countries, even to the other regions in the UK. Moreover, in that manual (book) the indirect flood damages are only available for some land use categories.

For a productive or service activity, the indirect loss due to disruption is the consequence of being unable to carry out the activity. This is caused by loss of or damage to the equipments and facilities, the failure of public utilities (communication,
electricity, gas, etc.), and the absence of the employees. Due to the disruption of the traffic connection, the products cannot be produced and/or delivered to the customers and its market share might be lost (temporally or permanently) to its competitors. The raw materials might get out of supply due to the limited stock capacity. If the users of the products of that plant cannot find other suppliers, they will suffer a secondary loss as well. If there are alternative traffic connections, the plant has to pay more transportation costs since a longer route must be followed.

A complete analysis of those indirect damages for a plant, which is very difficult even when the concerned plant under consideration cooperates well, proved to be possible (e.g. Penning-Rosell and Parker, 1987). However, no study on the indirect flood damages for a region on the basis of e.g. the damages for hundreds of plants, has been found in literature.

The value added published by CBS will be used, since in this study, the relevant definitions of CBS are adopted here. The value added is defined as "the difference between the production value and the consumption value". The production value and the consumption value is respectively composed by the elements mentioned in Table 6-1. Value added price consists of labour costs, gross result, and indirect taxes minus operating subsidies (CBS, 1990). This composition is also applicable to service sectors. The indirect flood damage refers to conditions of "no industrial operation or service". In an industrial production and service plant, no operation consequently implies no 'value added'. Therefore the total loss of value added of the plant in the period of flooding can roughly represent its indirect flood damage.

Table 6.1. Composition of production value and consumption value

<table>
<thead>
<tr>
<th>consumption value:</th>
<th>production value:</th>
</tr>
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<tbody>
<tr>
<td>- purchase of raw materials</td>
<td>- sales of goods produced</td>
</tr>
<tr>
<td>- changes in stocks</td>
<td>- changes in stocks</td>
</tr>
<tr>
<td>- consumption of energy</td>
<td>- fixed assets produced for own use</td>
</tr>
<tr>
<td>- other operating expenses</td>
<td>- benefit profit insurance</td>
</tr>
<tr>
<td></td>
<td>- margin on trading</td>
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<tr>
<td></td>
<td>- revenues from other non-industrial activities</td>
</tr>
</tbody>
</table>

The following indicates the steps to calculate the loss of value added during a flood:

(1) Insight in the duration of flooding (in days of disruption of activities) can be gained by investigating or simulating the physical characteristics of a flood event.
2. General information about the productive and service plants in the potential flooding area, e.g. the location, the number of employees and the activities, can be collected from local/regional commercial chambers.

3. The updated information about the production, consumption, value added, tax and subsidy, labour cost, employment volume, etc. for different societal sectors is available from the (national) Statistics Office. In the Netherlands, the information, at both national level and the provincial level is published periodically by CBS (e.g. CBS, 1991).

4. The average value added per employee per working day (or even per working hour, if needed) for different economic sectors can be derived from the information. The average value added per employee of a particular sector can be assigned to the plants in the study area which are valued in that sector. The loss of total value added can then be calculated on basis of the number of employees and the duration of disturbance.

6.4. Intangible Damage

6.4.1. Definition and content

Intangible damages at the individual household level include (Green and Penning-Rowsell, 1986; Green and Penning-Rowsell, 1989; Avakyan and Polyushkin, 1991-b):
- health effects, as a result of the physical effects (e.g. cold) as well as psychological effects (irritation, grief, the stress of experiencing a flood, worry about future flooding, sense of being unprotected, etc.);
- loss of memorabilia and other irreplaceable contents; and
- disruption to life: the problems and discomfort whilst trying to get the house back to normal after the flood, evacuation, etc.

Intangible damages at the community level have been defined as follows (e.g. Prakash, 1992):
- population at risk of being flooded;
- potential for loss of life;
- potential for destruction or disruption of communities;
- inconveniences due to infrastructure network and public utilities;
- damage to archaeological elements (objects), and other sites of cultural and historical values;
- damage to recreational facilities;
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- damage to fish and wildlife, wetland, natural vegetation and other environmental assets; and
- adverse political and institutional consequences.

Blocker and Rochford (1986) found in the study of a major flooding that, between one-half and three-quarters of flood victims suffer psychological and/or physical health impairment linked to the disaster. The major types of impairment were psychological damage, physical health, and disruptions in social relationships and daily living patterns. Symptoms included anxiety, depression, fatigue, discouragement, tension/irritability, nervousness, fear during storm, social isolation, sleep disturbance, increased use of medications, and increased visits to physicians. The study showed that flood victims made progress in alleviating their symptoms over time. Yet, as long as five years after the flood, a substantial proportion of victims still reported disaster linked symptoms.

The book by Gleser, et al. (1981) presents results of a very sophisticated investigation of the long-term psycho-social effects of flooding. Two years after a flood, an investigation was carried for about 600 survivors. In the book, the researchers clustered the damages of flooding into 3 summary scales: Anxiety (combining agitation, anxiety and stomatic concerns); Depression (combining depression, suicidal thoughts, social isolation, disruption of daily routine, and retardation of emotion); Belligerence (combining grandiosity, suspicion, belligerence, antisocial behaviour, alcohol abuse and speech disorganization). Moreover, they identified two primary scales: Drug Abuse and Overall Severity.

For sleep disruption, the study by Gleser, et al. (1981) found that two years after the flood, 77% of the males and 87% of the females reported that they 'sometimes' or 'often' had difficulty falling asleep. Over 75% of both man and women reported great difficulty in staying asleep. Nightmares were reported by nearly 70% of males and about 75% of females. Frequent use of sleep medication was reported by 17% of the men and 21% of the women, and reported usage tended to increase with age.

Blocker and Rochford (1986) stated that sex, age, educational level and presence of children are statistically significant factors related to the magnitude of intangible damages. Women had higher scores on the Anxiety, Depression and Overall Severity, but lower on Belligerence and Drug Abuse than men. Men with higher educational levels showed lower impairment scores on Anxiety, Depression and Overall Severity. Anxiety, Belligerence and Overall Severity were the highest in the age group 25-54.
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Anxiety was highest for men and women aged 40-54; Belligerence for those aged 25-39 and men 40-54 years of age. The presence of children in the household was related to higher impairment scores for men and married women, but not for the single, divorced, or widowed mother. It appeared that the ‘non-elderly’ or ‘middle-aged’ groups had a greater risk of impairment. Women reported more psycho-social impairment, while men reported more physical health symptoms.

The research by Blocker and Rochford (1986) did not support the hypothesis that those living alone would suffer greater psychological distress than those with spouses. Women who were living with their spouses at the time of flood were rated somewhat higher on Overall Severity and Belligerence than were the remaining women. Married men, however, were less likely to abuse alcohol than single or divorced men.

In conclusion, the magnitude and severity of the intangible damage depend upon three groups of variables:

1. The characteristics of the individual household affected (income, prior health status, personality type, social competency, insurance coverage, age structure, degree of social support, proportion of able-bodied adults in the household, and prior experience of flooding).

2. The characteristics of the dwelling unit affected (degree of flood proofing, susceptibility, e.g. basements, caravans or bungalows).

3. The characteristics of the infrastructure network and public utilities.

These variables can act synergistically. Thus an elderly disabled person living alone in a bungalow will be most seriously affected. Alternatively the variables may react antagonistically. A young couple will be less seriously affected and might possibly gain from flooding when they have a new-for-old insurance coverage; when they have savings and are socially competent. The many complex combinations of these variables imply that one can hardly predict the full effects of different types of floods on different households yet: a lot of uncertainty will remain.

6.4.2. Assessment methods

Although it is widely agreed that intangible damages should be integrated into the decision analysis process, there exists a disagreement about whether economic valuation of intangible damages is desirable (e.g. Coker and Richards, 1992). And for those who agree that the evaluation is desirable, the method of quantification remains problematic.
For the quantification of intangible damages at the individual household level, the following three approaches are available.

**Vulnerability analysis**
It is possible to describe approximately the vulnerability of different areas to flooding and the intangible damages of flood. Therefore, Penning-Rowsell and Green (1990) suggested to use surrogate variables as an alternative to the direct measurement of the household characteristics. The surrogate variables include those normally measured in flood alleviation feasibility studies (e.g. land use types, depth of anticipated flood, etc.) as well as those from the detailed population registration data (e.g. the percentage of population over 70 years of ages, percentage of single households, social class of head of household, etc.).

Evidently, these quantitative surrogate indicators constitute a rather indirect approach to the measurement of intangible damages. They cause a lot of uncertainty on whether the value(s) measured are in fact meaningful with respect to the real intangible damage. Moreover, for the quantification of damage assessment a lot of data, which meets the requirement of validity, completeness, etc, is required. These requirements can hardly be met in real life flooding situations. Consequently, one has to be satisfied with more qualitative measurements indicating that a particular (sub)area shows more (or less) than average vulnerability.

**Monetary equivalent method**
A second approach of quantifying intangible damages is an estimation of the monetary equivalent of 'intangibles'. The monetary equivalent assessment is based on the calibration of a partial model: equations which relate the depth of flood to the level of the direct losses, and in addition relate the level of these damages to the level of intangible damages (Green and Penning-Rowsell, 1986; Green, et al., 1987). Interviews or questionnaires are organized to let households assess the severity of some physical losses (e.g. damage to the house structure, loss to the replaceable content) as well as the severity of the intangible aspects. When the severity of the physical loss is of the same magnitude as that of intangible aspects, the monetary values of the physical losses are supposed to be equivalent to those intangible loss values.

Obviously, when two physical losses have the same severity as that of intangible aspect but different loss values, it remains problematic which of the two physical loss should be assigned to the intangible loss.
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Willingness to pay
Since the U.S. Water Resources Council, the Army Corps of Engineers and other agencies have sanctioned the use of Contingent Valuation methodology in cost-benefit analysis, surveys have been used with increasing frequency to obtain estimates of the benefit for public goods, such as water quality (Lindsey, 1994).

Thunberg and Shabamn (1991) found from a survey of landowners residing in a floodplain that benefit in reducing non-property damage is one of the overall benefits, and more specifically, benefits of the mitigated community disruption are more important than that from reduced psychological stress among floodplain occupants. It was also found that, although only 10% percent of the city’s population lives in the floodplain, 56% percent voters favoured the tax increase to support the flood reduction project.

Recently several surveys on the public affected were carried out to estimate the willingness to pay for water supply and treatment service (Green, et al., 1993; Platt and Piper, 1994; Bonrud, 1994). Obviously, surveys of the households which recently experienced a flood can help to illustrate the willingness to pay for the intangible flood damage. However, no such a survey is found in the literature.

6.5. Human life loss issue

Discussion of the potential failure of water projects inevitably raises the issue of human safety and the value of life. Although the numbers of deaths in extreme floods is relatively low in comparison with death due to auto accidents or industrial accidents, the loss of a number of human lives in a single event heavily influences the societal perception of (un)safety.

6.5.1. Human life loss assessment

Death may occur in floods through a number of mechanisms: collapse of the building in which people are sheltering, exposure, drowning whilst being in a building, and drowning whilst attempting to wade through flood waters.

The attempts to estimate loss of life from flood events can be divided into four types: empirical studies—use surrogate variables such as the structural damage of floods for
population at risk (e.g. Petak and Atkinson, 1982); theoretical models—employ best-guess parameters that are derived from the historical record of dam failure and flash flood (e.g. Dekay and McClelland, 1993); models that utilize historical data and other investigated information (Chatterton, et al., 1993); and estimation models based on information such as the velocity of the flood water, flood depth, and weight of human objects obtained from experiments (e.g. Abt, et al., 1989). Different approaches need different information.

Petak and Atkinson (1982) used surrogate variables such as the structural damage of floods for the population at risk.

DeKay and McClelland (1993) developed a theoretical model based on the historical record of dam failure and flash flood cases via logistic regression. In this study, the loss of life was expressed as the function of warning time, the size of the population at risk, and the forcefulness of the flood water.

Chatterton, et al. (1993) introduced a conceptual formula for the estimation of the number of deaths as a consequence of building failure can be estimated as:

\[(\text{number of buildings}) \times (\text{probability that a building will fail to provide protection}) \times (\text{probability of death should the building fail}) \times (\text{time weighted occupancy})\]

The probabilities in this equation are very difficult to estimate. No application of the equation has been found in the literature.

To predict the instability of (human) objects in flooding, Abt, et al. (1989) conducted an experiment to identify when an adult human could not stand in a simulated flood flow. They examined the stability on a number of monoliths and 20 human objects, and built a model for the relationship between the product of the weight and height of the object and the point of instability of the object.

In principle, the prediction of the loss of life in flooding has not yet reached a satisfactory level. The methods described above either failed in successfully considering the non-homogeneity of historical data or in considering the human’s real responses to flood warning.
Flood damage assessment

6.5.2. Human life value assessment

Potential losses of life are likely to be a critical issue in the decision analysis process. However, regardless whether or not one wants to consider the value of a life, apprenenent of any project which might result in life losses in case of failure (such as a nuclear power plant), or which aims to reduce the potential number of life losses (such as a flood alleviation project), implies a human life valuing process. The issue of life value cannot be avoided (Baecher, et al., 1980; Zhou and Van der Heijden, 1994-c).

The number of expected deaths with and without a protection work should be compared to determine the reduction or increase of expected fatalities, which can be regarded as one kind of the project consequences. The value of life which the society, or the majority of the society’s members want to accept is implicitly or explicitly defined in the decision making process.

An implicit analysis to deal with life values leads to the following. Let $B_1$ be the benefit, $C_1$ be the cost of a project, and $F_1$ be the expected increase of loss number of the human life in comparison with the situation without the project in case a potential project (e.g. a dam) fails. In case of a positive net benefit, the acceptance of the project implies the acceptance of the following quantification of life value:

$$\text{life value} < \frac{[B_1 - C_1]}{F_1}$$  \hspace{1cm} (6-1)

Similarly, when the project is not acceptable, then:

$$\text{life value} > \frac{[B_1 - C_1]}{F_1}$$  \hspace{1cm} (6-2)

When the net benefit is negative, the value of human life is not implied.

For a flood alleviation project, in case the net benefit is negative, let $B_2$ be the reduced flood loss, $C_2$ be the cost, and $F_2$ be the reduction of the number of the potential life loss due to the flood alleviation project as compared to the do-nothing situation. The acceptance of the project implies the acceptance of the following quantification of life value:

$$\text{life value} > \frac{[C_2 - B_2]}{F_2}$$  \hspace{1cm} (6-3)
Similarly, when the project is not acceptable, then:

\[ \text{life value} < \frac{[C_2 - B_2]}{F_2} \quad (6.4) \]

When the net benefit is positive, and the expected number of life loss is reduced with flood alleviation measures, the project is acceptable, but no life value is explicitly quantified. However, when one flood alleviation alternative is preferable to others, an implicit value of life loss is assigned.

In case life value is explicitly brought into discussion (in a quantitative way), the figures used are generally based on normative or empirically determined standards. The most commonly used economic value of human life is the so-called gross production method. It is the present value of the expected gross income which the victim could make in the future.

In the USA, figures between 200,000 and a million US$ per human loss are considered acceptable for decisions in the transportation sector (Baecher, et al., 1980). In Sweden money values between SKr 200,000 and 280,000 (in 1986 SKr) were ascribed to accidents respectively in urban and rural areas (Report 1). In Denmark, the cost of an accident with personal injury is described as a million DKr per life (in 1989 DKr) (Report 1). In the Netherlands, the average value of a traffic accident victim is Fl. 600,000 (in 1979 Fl.) (De Mol, et al., 1987). According to Fernandes-Russell et al. (1988), referred by Soby, et al. (1993), study of the retrospective value of life as implied by safety investment decisions in various sectors in the UK finds that implied values of a life range from 200,000 to 4 million £ (in 1986 £).

6.6. Magnitude relationships among the various kinds of damages

The total flood damage is the summation of the tangible and intangible effects. If the intangible damage and the human life loss would be measured in monetary terms, they can be directly added to the tangible effects, and the total flood damage can be measured in monetary units.

The ratio of indirect to direct flood damages depends on the category of the property affected as well as the flood depth, duration, etc. Generally, the ratio is less than 1 : 10. For some highly specialised and concentrated industries it can be as high as 1 : 3 (Penning-Rowsell and Green, 1990). Unfortunately, it is not possible to identify those
Flood damage assessment

industrial properties with a high vulnerability without undertaking a survey of the properties concerned. Since the indirect damage includes clear-up costs and the cost of clearing does not vary greatly with increased depth of flooding, the ratio tends to decline—often markedly—with increased depth of flooding.

Industrial properties show a wide variation in the level of indirect flood damages. There is however much less variation in the ratio of indirect to direct damages, because the duration of production stops or reduced production tends to be associated with the severity of the direct damage.

The tangible direct damages are mainly estimated from flood depths and durations. These estimates often form the basis of further rule-of-thumb estimates of indirect damage. For example, Kates (1965) analyzed a number of studies by the Corps of Engineers and found the indirect damages to residential units to be 15% of direct damages.

Intangible damages may be comparable in importance to tangible damages in developing disaster mitigation and assessment procedures. Intangible damages to residences have been estimated as 112% of direct damages (Johnson, 1976). However, the literature reports few attempts to consider intangible damages in the formal decision analysis.

Recent studies indicate that in some circumstances, especially where flooding is not accompanied by a warning and is relatively severe (as in coastal locations, or sudden thunderstorm events), intangible damages can be at least as important, if not more important, as tangible damages (Green and Penning-Rowsell, 1989; Penning-Rowsell and Green, 1990). This is an important finding since it means that we do not yet adequately account for the full benefit of flood alleviation in the benefit-cost comparisons and tend to account for just what is easy to measure rather than what is important.

The severity of various kinds of damages are related with each other. Therefore, with such relationship(s), some kinds of flood damages which are usually difficult to estimate may be estimated on the basis of knowledge on other kinds of damage.

6.7. Conclusions

Among various assessment methods for different flood damages, specific ones can only
cope with specific types of damages. These available methods tend to account for just what is easy to measure rather than what is important and they do not yet adequately capture the full benefits of flood alleviation projects in cost-benefit comparisons. Much attention has been paid in this chapter to the development of the assessment methods which are applicable at a regional level. An overview of the assessment methods for flood damage is presented in Table 6-2.

Table 6.2. An overview of assessment methods for various kinds of flood damages

<table>
<thead>
<tr>
<th>types of flood damage</th>
<th>assessment methods</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>for households &amp; companies</td>
</tr>
<tr>
<td>tangible direct damage</td>
<td>unit-loss</td>
</tr>
<tr>
<td>tangible indirect damage</td>
<td>value added</td>
</tr>
<tr>
<td>Intangible damage</td>
<td>vulnerability analysis</td>
</tr>
<tr>
<td></td>
<td>monetary equivalent</td>
</tr>
<tr>
<td></td>
<td>willingness to pay</td>
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</tbody>
</table>

Flood damage assessment has made progress in the last decades. Assessment methods and loss values for tangible direct damages have been developed; the issues about the tangible indirect and intangible damages have been addressed intensively in the literature. It has always been recognised that direct and indirect damages that could be measured in monetary terms were not the only—or indeed not necessarily the main—damages of floods. This topic, however, has not been studied sufficiently.

In practice, the unit-loss method has been widely used in flood alleviation studies. Nevertheless, a limited variety and quality of methods are revealed. Their applicability is limited by data requirements and assumptions on relationships, key values, etc.

The unit-loss method will be applied to assess the tangible direct flood damage in the application of this study in which the potentially flooded properties will be inventoried with the aid of ARC/INFO GIS. The Value Added method will be used to roughly estimate the tangible indirect damage associated with the productive and service disruption. The magnitude relationship of tangible direct to intangible damages identified in the literature will be used to indicate the range of the intangible damages. The application of these methods can be found in Chapter 8.
CHAPTER 7

UNCERTAINTY ASSESSMENT FOR DECISION ANALYSIS

7.1. Introduction

Various kinds of uncertainties occur in water related engineering projects during the planning and design, the construction, and the operation and maintenance stages. Uncertainty can be regarded as an important but also difficult to cope with issue at those different stages.

Methods to incorporate risk and uncertainty assessment in water resources engineering are documented in the literature. The most often used methods include: sensitivity analysis, interval analysis, fuzzy set theory, and a probabilistic design approach. Different techniques are suitable to handle uncertainty information of different kinds: if no information on the uncertainty magnitude is available: sensitivity analysis; if the magnitude is only known approximately: fuzzy set theory; if the upper and lower limit is known: interval analysis; if the probability distribution of uncertainty is known: a probabilistic design approach.

There is a wide range of literature on the application of uncertainty analysis in the water related fields. Meynink and Brady (1993) studied the effects of the uncertainty associated with the estimation of catchment response parameters in a rainfall-based method of flood estimation. Bao, et al. (1987) analyzed the significant effects of uncertainties in a flood magnitude estimation on annual expected flood damage. Afshar and Mariño (1990) incorporated uncertainty in the estimation of flood magnitude in optimization of spillway capacity and concluded that the uncertainty effect resulted in a major change in the optimum spillway design capacity. Tung, et al. (1993) assessed the hydrologic uncertainties and incorporated them into the evaluation of overtopping risk and other related measures of a flood detention reservoir. Zhou (1994-a) investigated the influence of the hydrologic uncertainty associated with the extrapolation of the boundary condition on the design water level in the river transition zone. Some work on uncertainty analysis of hydrologic inputs was included in Ganoulis (1991-b).
Singh and Melching (1993) studied the hydrologic uncertainty in the design discharge and the hydraulic-model uncertainty in transforming the design discharge into a flood stage. Afshar, et al. (1994) examined the hydrologic and hydraulic uncertainty in the estimation of the flood magnitude and incorporated them into an optimization model for river diversion. Mays (1979) showed, with illustrative cases, that in addition to the hydrologic uncertainty, other kinds of uncertainty such as those in hydraulic modelling and in cost also have an effect upon the design of highway drainage culverts.

Karamchandi and Cornell (1992) examined the sensitivity of failure probability estimates to changes (uncertainties) in distribution parameters. Melching, et al. (1990) evaluated the combined effect of the uncertainty in data, model parameters and model structure in a watershed runoff model with probabilistic techniques (Level 2 and 3). Song and Brown (1990) analyzed the influence of the uncertainty in correlated input data on the output of the Streeter-Phelps dissolved oxygen equation with the techniques of sensitivity analysis and probabilistic approach at Level 2 and 3. Lee and Dahab (1992) studied the uncertainty in the nitrate risk-management process with fuzzy set theory.

When the influence of uncertainty on the failure mechanism is analyzed, the reliability function, in which the reliability of a certain failure mechanism is expressed as a function of its causative variables, should be established. TAW (1990) established a reliability function for dike overflow in the coastal zone, whereas Zhou (1994-b) further developed a reliability function for the river transition zone.

Up to now, water resources systems have been designed under the fundamental assumption that the hydro-meteorological inflow process is cyclic-stationary in nature, i.e. that the stochastic properties of the process, which one can identify by inspection of the past records, will be reproduced unmodified in the future, when the system will be in operation (Soncini-Sessa, 1994).

While uncertainty can be used as a sword or shield (to justify a policy or to avoid decision), it remains useful to characterize it and determine whether the qualitative and quantitative means to cope with uncertainty can lead to more confident decisions, particularly when costs can be extremely high and the benefits might only accrue long after a decision has been made (Cox and Ricci, 1994). Uncertainty analysis should thus focus on the influence of the individual components and the overall uncertainty on the outcome of decision analysis.
Uncertainty assessment

In this chapter, various uncertainties in the decision analysis process in flood alleviation focusing on dike heightening will be identified; methods for uncertainty analysis will be introduced, and practical set-up of uncertainty analysis will be discussed.

7.2. Uncertainties on individual components

In water resources engineering, uncertainty arises from the inherent uncertainty of the climate and weather conditions. Uncertainty is associated with the spatial and temporal variability of the hydrologic variables. Uncertainty is also related to the use of methods and tools to sample, describe, model and predict the phenomenon which influences the evaluation of the performance and cost of the design, and consequently influences the design concept. This uncertainty forms an important component of design based on a risk assessment approach which incorporates especially consequence assessment.

As mentioned in Chapter 4, three types of uncertainty about a system are involved: intrinsic, modelling and measuring uncertainty. Hydraulic designs are subject to all of these uncertainties.

In this section, the uncertainty associated with the failure mechanism of overflowing, hydrologic modelling, the timing of boundary conditions, dike construction, hydraulic modelling, inundation depths, flood damages, cost and adverse impacts of dike heightening, and socioeconomic development will be discussed.

7.2.1. Uncertainty on failure mechanism associated with overflow

As described in Chapter 2, it is at present not possible to predict accurately the dike overflow and breaching process and consequently the flooding stages in the polder when the water level exceeds the dike crest. This uncertainty is due to the insufficient knowledge on the failure mechanism of dike breaching. Modelling and measurement uncertainties are associated with the estimation of the failure mechanism.

7.2.2. Hydrologic uncertainty

Hydrologic uncertainty is associated with prediction of hydrologic events such as stream flow and rainfall (Bao, et al., 1987). Hydrologic variables depend on meteorologic conditions and physical circumstances in a river basin. Hydraulic and land development
projects, and forestation and deforestation can change physical conditions of the flow of water in river basins. Other human activities can indirectly influence the global as well as regional climate.

Hydrologic uncertainty contains inherent-, modelling- and measuring components (e.g. Mays, 1979; Rasmussen and Rosbjerg, 1989). Hydrologists have long been concerned with the problem of estimating the flood discharge at a gauging site of which the return interval is much longer than the period of record (e.g. Lettenmaier and Potter, 1985). Uncertainty may arise from the interpolation and/or extrapolation of the probability distribution of hydrologic variables, non-homogeneity in samples as well as in population caused by non-stationary climatic conditions, changes in catchment conditions caused by human activities, errors in data gathering and insufficient hydrologic data. The estimate of the magnitude of an extreme flood is highly dependent on the selected probability distribution (e.g. Parakash, 1989).

Practices of dealing with various hydrological errors or uncertainties due to data sampling are reported by e.g. Rasmussen and Rosbjerg (1989), Chowdhury and Stedinger (1991), Rasmussen (1991), and uncertainties due to modelling are described by a.o. Duckstein and Bogardi (1981) and Enzel, et al. (1991).

7.2.3. Uncertainty in the timing of the peak for boundary conditions

In addition to the uncertainty associated with the extrapolation of boundary conditions, the lack of information on the timing of the peak discharge at the upstream and the peak sea level will significantly contribute to the uncertainty in the estimation of the exceedance probability of overflow volume in polders. Inherent-, modelling- and measurement uncertainty is associated with this timing. As described in Chapter 5, further research on this issue is required to improve the estimation of flood magnitude and the exceedance probability in the transition zone.

7.2.4. Uncertainty on dike height

Given a design dike height, dike construction is not very precise; variations in height of 10 centimetres over the length of the dike body have been normal in the past (Van der Kleij, 1977). Advanced measurement technology allows nowadays to minimize the uncertainty due to inaccurate construction.
Uncertainty assessment

In addition, after construction, dike height can decrease due to soil settlement.

7.2.5. Hydraulic uncertainty

Hydraulic uncertainty is associated with the determination of flow capacities for hydraulic structures. This arises from the use of the simplified description of the natural physical phenomena by mathematical models, imprecise data on hydraulic structures, non-uniformity of construction materials, and various operational conditions (Bao, et al. 1987; Singh and Melching, 1993; Afshar, 1994). Some hydraulic parameters calibrated in an open channel flow model—such as Chezy roughness coefficient—are less reliable for high flow situations than those in the normal data range of model calibration. The hydraulic uncertainty involved in flood simulation is therefore expected to be larger than for calculation of average flow.

7.2.6. Uncertainty in inundation depth

As mentioned before, given the amount of overflow water, the inundation level in the polder can be derived from the topographic characteristics of the polder. However, soil penetration, seepage to ground water and evaporation from the flood water can also significantly influence the actual inundation level. Seepage and soil penetration depend on antecedent conditions, the type of the soil and its use as well as the saturation of the soil in the course of flooding. These influences vary within different polders and time of flooding and are thus difficult to estimate. Modelling and measurement uncertainty is associated with the determination of inundation depth for a given overflow volume in polders. A rainfall-runoff model usually aims to deal with this type of problem. The inherent uncertainty on the moisture status of the polder at the onset of the flooding remains in any case.

7.2.7. Uncertainty in flood damages

Uncertainty in flood damages results from incomplete information on the properties in the potentially flooded floodplain. This can originate from a biased or inaccurate estimation of flood loss values of individual property, the incomplete information on the relationship between the industrial & service plants in the floodplain and related plants outside of the floodplain, and ineffective evaluation methods for indirect and intangible damages. For the given physical characteristics of a flood, both modelling and measurement uncertainty are involved in flood damage assessment.
Chapter 7

Uncertainty in assessing direct damages
The assessment of the tangible direct flood damage requires two sorts of information: the inventory of the properties in floodplain and the economic value of each kind of property. Both types of information have uncertainties associated with them.

Inventory uncertainty decreases with the level of detail. GIS allows to improve greatly on the level of detail by strongly increasing the processing capacity. Flood loss functions have averaging errors because they are not applied to each individual component but to a class of components.

As a result of the substantial improvement in the registration of spatial objects, information about the properties in the floodplain, as well as in the non-floodplain, can now be provided in the form of digitized maps. That makes it possible to apply modern computer tools such as GIS for the flood damage assessment. The digitized map can greatly reduce the magnitude of the uncertainty associated with property inventory. Nevertheless, some uncertainties remain.

Remaining uncertainty associated with application of GIS
Uncertainties and errors are present in GIS. These errors and uncertainties have been classified into four types (Bedard, 1987): conceptual uncertainty, spatial uncertainty, attribute uncertainty and media uncertainty. Clearly these four kinds of uncertainty are subjected to the modelling and measurement uncertainty classified earlier.

Conceptual uncertainty focuses on the problems of translating the entities, states and problems which exist in the real word into a sequence of physical and cognitive models which form the basis of any GIS project.

Spatial and attribute uncertainty are functions of location and attribute errors associated with a given data set, and reflect the users' concern over the quality of both the spatial and descriptive information.

Media uncertainty refers to the degree to which the other types of uncertainty are known to the user.

It would be impossible to develop a model which can handle all of these uncertainties as some of them are simply due to a lack of planning and knowledge of the systems by individuals concerned.
Uncertainty assessment

However, all of the GIS users have one thing in common: uncertainty over their data. The primary source of data uncertainty is the level of errors contained within a spatial data set. The errors most likely to be present in spatial data can be grouped into the following categories (Coward and Heywood, 1991): lineage, positional accuracy, attribute accuracy, logical consistency and completeness.

Lineage errors are those which are inherent in the source material from which the digital product has been derived. They are often the most difficult to identify since few if any spatial data products have traditional carried an accuracy statement.

Positional accuracy is the different between the true position and the estimated position of a feature.

Attribute accuracy is concerned with the characteristics of non-positional information, for example whether a specified soil type on a map is found at its co-ordinate location.

The errors associated with logical consistency are characterised by the degree to which the data structures employed by the GIS to model the real world actually replicate the features they have been designed to represent.

Errors of completeness are characterized by whether a given data set contains all the elements it claims to. For example, if a data set consists of a region’s river network, does it contain all the rivers in that region, or only those of major importance.

In principle, there is no effective way yet to assess the uncertainty due to the application of digitized land use maps.

Uncertainty in assessing indirect and intangible damages
The indirect damage estimation for a certain plant (e.g. a company) requires to consider various information such as who supplies the raw material, who uses its products, and where its employees live. Part of the information is quite often not accessible due to confidentiality.

Although some studies have dealt with intangible flood damages (e.g. Green and Penning-Rowsel, 1986), there is still no widely acceptable or effective method to deal with this issue, let alone to accurately assess the intangible damage.
The large uncertainties in assessing indirect and intangible flood damage are associated with both the information and assessment models.

When some of the flood damages—for example the direct ones—can be assessed, then it may be possible to roughly estimate the total damage if a relationship between the magnitudes of direct, indirect and intangible damages can be established. A large uncertainty will however remain.

**Uncertainties associated with human life losses.**

In addition to the uncertainty on the potential number of human lives lost in flood, the discussion on potential floods inevitably raises the issue of "the value of life". Although a lot of researchers suggest to separate the economic loss from the life loss in the final decision, the issue remains unavoidable. There appears to be no agreeable method yet to value a human life.

**7.2.8. Uncertainty on the cost**

Implementation of dike heightening requires investment and has adverse social and environmental impacts. Uncertainty is associated with the estimation of each of them.

Estimation of the cost involves uncertainty due to changes and/or variations in discount rate, building technology and construction methods and the regional differences in the cost of materials and labour.

If environmentally friendly designs are applied, then the design may change and the construction cost associated with such a design will probably increase. Such a design should take into account the local environments; uncertainty is introduced if decision analysis cannot consider all the details of the dike environment. Modelling and measurement uncertainty influence the estimation of the dike heightening cost.

**7.2.9. Uncertainty due to the socioeconomic development**

Uncertainties are associated with the estimation of the values to be protected which change over time and depend on the rate of economic development. Those factors have large modelling and measurement uncertainty due to the unforeseen socioeconomic development in polders and an uncertain economic growth.
Uncertainty assessment

There are further changes in the perception of the intangible flood damage and the social and environmental impact of dike heightening.

7.2.10. Overview of uncertainty components

The various uncertainties, which have to take into consideration in the decision analysis for dike heightening as discussed above, are presented as solid boxes in Fig. 7.1.

Fig. 7.1. Uncertainties in the decision analysis process for dike heightening

7.3. Characterization of uncertainty analysis

Uncertainty can be expressed by an absolute value range or a probability distribution. Different expressions of uncertainty involve different methods for uncertainty analysis. Uncertainty on the outcome of a complex phenomenon will be a composite of the uncertainties of individual factors or components. Expressions of uncertainty for
individual components may be more easily derived while an uncertainty assessment for a complex phenomenon will always depend on a tracing of the individual uncertainties of its components. In this section the total uncertainty of a complex phenomenon like dike heightening and the role of uncertainty in decision analysis will be discussed.

7.3.1. Methods for uncertainty analysis

Several methods for dealing with uncertainty are available in the literature. The most often used methods include: sensitivity analysis, interval analysis, fuzzy set theory, probabilistic design. Different techniques are suitable for different kinds of uncertainty information and different systems.

An uncertain parameter, such as a discharge with a certain exceedance frequency, or an expected flood depth, can be considered to take on a value within a certain interval. With more information on the uncertain parameter, or a better method to reduce the individual uncertainty, the interval can be "narrowed". With more information about the outcomes of the uncertain process, a probability distribution may be assigned to the interval expressing e.g. a higher likelihood for the middle of the interval.

Uncertainty analysis for a decision problem can be defined as the study of the propagation of the uncertainty in the input variable through the decision model resulting in an uncertainty over the value of the output. In this study, the output is the net benefit for a particular dike heightening alternative. Several methods to estimate the uncertainty on the overall output are discussed below.

If no information on the magnitude of uncertainty is available, then sensitivity analysis can be applied. Sensitivity analysis investigates the effect of changes in input variables on output variables. Usually, a percentage variation of a parameter in the decision model is considered, while the other variables remain constant. Sensitivity analysis shows the influence of this variation on the final outcome of decision analysis. By comparing the influences of different parameters in the decision model, one can establish the relative contribution of the individual uncertainty to the outcome of decision analysis. Sensitivity analysis can identify the individual uncertainties which have the most important contributions to the uncertainty on the final decision.

Interval Analysis is the simplest way of considering uncertainty (Duckstein, 1994). Interval Analysis can be applied if the upper and lower limit of the parameter to be
Uncertainty assessment

estimated is known. The difference between the Sensitivity Analysis and Interval Analysis is that the first method indicates the relative influence of the individual uncertainty, whereas the second illustrates how large the influence of the individual uncertainty can be.

If the magnitude of uncertainty is not well known and/or not correctly measurable, fuzzy set theory can be applied (e.g. Kaufmann, 1986). For example, the values of the basic variables can be estimated as fuzzy numbers to characterise their uncertainty.

If the probability (distribution) of uncertainty can be estimated, a probabilistic approach such as the techniques at Level 2 and 3, in particular the Monte Carlo simulation method is an effective technique when some types of uncertainty can be expressed with numerical probability distributions.

7.3.2. Practical set-up of the uncertainty analysis

As illustrated in Fig. 7.1, a large number of causes of uncertainty can be identified for dike improvement decision analysis. Not all of these causes are equally important and the degree of accurateness in the estimation of the uncertainty ranges varies considerably due to the difference in available information.

A drawback of the interval analysis method for uncertainty assessment, which will be applied to the application presented in Chapter 8, is that there is no indication of preference among the values in the interval. This may lead to a very wide range of uncertainty for the output variable without any indication of the likelihood of values within the range. This may complicate decision analysis by diffusing trade off.

By including or excluding certain causes of uncertainty some (subjective) judgement on the importance of this uncertainty and the information on which it is based can be expressed. Inclusion of all (many) possible sources of uncertainty may further lead to a cumbersome analysis without adding substantial information to the decision analysis process. In the application a total of four sources of uncertainty have been retained, those are: the uncertainty in the dike overflowing mechanism, the uncertainty due to the use of different probability distribution for the extrapolation of the hydrologic variable at the boundaries, the uncertainty in the assessment of flood damage and the uncertainty in the estimation of dike heightening cost. The influence of these uncertainties on the decision analysis for dike heightening will be examined.
7.4. Conclusions

Various uncertainties in the decision analysis process in flood alleviation have been identified.

General methods for uncertainty analysis have been introduced in this chapter. Through studies on decision analysis under uncertainty, the influence of different kinds of uncertainty on the decision on dike heightening can be analyzed. Uncertainty analysis provides useful information for decision analysis. Being aware of various uncertainties, any decision on dike heightening must be made under uncertainty.

In the next chapter, methods operationalized in this part will be applied to polders in the river transition zone in the Netherlands. Four kinds of uncertainty mentioned above will be examined in the application. The uncertainty in the inundation level in polders due to the soil penetration and seepage, and uncertainty in the timing of peak boundary conditions might be also important, but will be not included in the application due to the lack of information.
Part VI

Application
CHAPTER 8

APPLICATION TO THE POLDERS
IN THE RIVER TRANSITION ZONE
IN THE NETHERLANDS

8.1. Introduction

To illustrate the applicability of the risk assessment framework proposed in Chapter 4 and operationalized in Chapters 5 to 7, a real world application has been carried out to a system of four polders in the transition zone in the Netherlands. The study focuses on flood alleviation by means of dike heightening.

In this chapter a description is given of the application area followed by an optimization of the flood protection for the four polders. The optimization proceeds in two steps: first an optimization for the combined system of the four polders, followed by a further refinement of the optimization by searching for the optimal protection level for each individual polder using a gradient search.

Assessment of uncertainty on the various parameters of the decision analysis process indicates a very large uncertainty on the net benefits of alternative protection strategies; careful evaluation is therefore required.

8.2. Description of the application area

The application area is located in between the river Waal and Meuse (Fig. 8.1). It comprises four neighbouring polders, each surrounded by dikes that make up their boundaries, with a total surface area of about 500 km² and a total population of about a quarter of a million. The four polders are: "Land van Heusden en Altena, de Oostwaard"; "Land van Heusden en Altena, de Noordwaard"; "Eiland van Dordrecht"; and "Hoekse Waard". According to the present numbering convention of the 53 Dutch polders (Bakker, 1989), they have received, respectively, the number 24, 23, 22 and 21.
Fig. 8.1. Schematization of the river network in the application area

For this area a substantial amount of data is available, including:
- hydrologic data at Lobith, Hoek van Holland, Tiel and Lith; and
- socioeconomic information such as digitized land use maps.

A software of one-dimensional model of open channel flow called ZWENDL, developed by Rijkswaterstaat, is used in this application. It calculates the behaviour of basic hydrologic variables in river stretches, based on scenarios which have been formulated as input. All hydrologic variables such as water level, speed, and flow are described to be same in a river cross-section, but differ with the site along the river and with time (Struijk, 1992). The program containing bifurcations is applicable in a river system or estuary. To calculate the overflow discharge at dike sections, some artificial river sections and a sluice have been appended in the river schematization of ZWENDL to each of the dike sections where overflowing is considered in this application (refer to Chapter 5).

The numbering of the river sections in this application is presented in Fig. 8.2.
8.3. Formulation of dike heightening alternatives

Alternatives for dike heightening are usually formulated in correspondence with the probability distribution of the water level at the site of interest along a river (local water level). The probability distribution of the local water level can be estimated from the probability distribution of hydrologic variables at boundaries.

There are two steps in this estimation process: (1) establish the relationship of the discharge at Lobith and the sea level at Hook van Holland with the water level at the site of interest; (2) transfer the probability distribution (or numerical curve) from the boundaries to the local water level using this relationship. With the probability distribution of the local water level, given a proposed protection level in terms of probability, the corresponding local water level can be derived from the probability distribution of the local water level. The local water level corresponding to a protection level is called the local design water level. At all locations of interest along a river, the probability distribution of the local water level can be constructed in the same way. Given a proposed protection level, all local design water levels can be derived accordingly. Heightening of the existing dike, up to the local design water level, can protect the polder up to the proposed protection level. In this way, alternatives of dike heightening can be formulated.

In the river transition zone, it is not practical to formulate alternatives for dike
heightening in such a way for the following reasons. Firstly, since water levels in the transition zone are dependent on both upstream discharge and the sea level, it would cost a lot of time to establish the relationship of hydrologic variables at the up- and downstream boundary and at specific locations in the transition zone. Secondly, any dike heightening (alternative) would change these relationships, thus for each dike heightening alternative the relationship at each location should be reconstructed accordingly. Thirdly, using such relationships, the probability distributions of the water level at all locations should be integrated as described in Chapter 5, which is time-consuming.

To avoid this complication in the application, the protection levels have been formulated on the basis of a specific combination of boundary conditions. The dike height corresponding to a proposed protection level is then derived as follows: (1) given a specific boundary condition combination (an upstream discharge and a downstream sea level), the corresponding water levels at different locations in the application area are calculated; (2) the proposed dike height corresponding with a proposed protection level is formulated by heightening the existing dike up to the water level, so that dike overflowing will not occur when the proposed dike heightening is completed.

The Rhine and the Meuse rivers jointly influence the flow situation for the four polders. The discharges at Lobith (Rhine) and at Borgharen (Meuse), entering the zone of interest, belong to different upstream river basins but are not fully independent from each other. The two systems should however be considered only loosely coupled as the interaction during flood conditions will consist mostly of back water effects on the links between the two rivers.

In earlier considerations on flooding by Rijkswaterstaat, flooding situations on both rivers have been decoupled and a fixed (average) flow is considered for one river while considering flood conditions on the other. This approach is also followed in the present application. Consideration of a third boundary condition would magnify the number of boundary condition combinations to be considered and strongly complicate the estimation of the joint probability between the different boundary conditions.

A set of 16 alternative protection levels corresponding to 16 boundary condition combinations is considered. The 16 alternatives in function of the boundary condition are presented in Fig. 8.3. The graph can be read as follows: Alt-01 (Alternative 01) corresponding to $Q_{Lob}=9900$ and $H_{HVH}=4$ indicates that this alternative can protect
polders from a flood condition caused by $Q_{\text{Lob}} = 9900 \text{ m}^3/\text{s}$ and $H_{\text{HvH}} = 4 \text{ m}$.

![Diagram showing protected levels for each polder]

**Fig. 8.3. Proposed protection levels for each polder**

Let $M_j(Q_{\text{Lob}}, H_{\text{HvH}})$ represent the measure for polder $j$ to protect this polder against the situation arising from the discharge at Lobith ($Q_{\text{Lob}}$) and the sea level at Hoek van Holland ($H_{\text{HvH}}$), for example $M_2(16500,5)$.

The present situation, the "do-nothing" alternative, provides the basis for comparison of the 16 alternatives.

As mentioned in Chapter 4, if the protection level is allowed to be different for different polders, there would be $16^4$ dike heightening alternatives in this application. To tackle the optimization over such a large set of possibilities, an approach in two steps is proposed. In a first step the four polders will be regarded as one combined system and an optimum level of protection for the combined four polders is determined. This is followed by a differentiation of the protection levels for the individual polder using the overall level of protection as a starting point. This represents a further (more detailed) search for the optimal level of protection for the total system with free selection of protection levels for each polder in correspondence with benefits and costs for each
individual polder.

The concept presented in Chapter 4 is applied here with a modified formulation. The overall levels of protection for the combined system of polders are here formulated in terms of the protection for a particular combination of the boundary conditions as defined above.

The optimization will then progress as follows:
- determination of the flooding performance (net benefit) for each of the 16 alternatives; and
- based on the identified best solution for the combined polders (one of the 16) a further search is performed to differentiate the individual polders based on a gradient search to further increase total net benefits.

To upgrade the existing dike heights to the level corresponding with any of the 16 alternatives, the water level associated with the alternative for each polder should be determined first. The water level of the alternative, as described in Chapter 5, is the resulting water level under the boundary conditions corresponding with the alternative and the maximum interaction between the boundary conditions.

The numbering of the dike heightening alternatives should not be interpreted as a ranking in terms of protection level. The protection level associated with each of the alternatives has a spatial implication for the different polders. The interpretation of the different protection levels should be made according to the diagram in Fig. 8.4 which illustrates conceptually the dike height gradients in a spatial context. A number of alternatives is illustrated in the diagram. For example, Alt-11 has a higher protection level than Alt-05 and Alt-16 higher than Alt-15. However, Alt-11 and Alt-16 are not comparable with each other: Alt-11 has a better protection from the discharge of the Rhine than Alt-16, whereas Alt-16 has a better protection from the sea than Alt-11.

The dike heights at some sections in Alt-05 and Alt-13 are presented in Fig. 8.5. The dike heights in Alt-05 are higher in the upstream part and lower in the downstream part in comparison with Alt-13, since Alt-05 should protect from flooding resulting from the discharge of 16500 m$^3$/s at Lobith and the sea level of 4 meter at Hoek van Holland, while Alt-13 from the discharge of 9900 m$^3$/s and the sea level 6 meter.
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Hoek van Holland

6 m  Alternative 16  16500 m³/s
5 m  Alternative 11  14850 m³/s
4 m  Alternative 05  13200 m³/s

Lobith

Fig. 8.4. Illustration of dike height gradients for different alternatives

dike height (m)

Fig. 8.5. Dike heights before and after heightening

With respect to the gradient search carried out in the second step of the optimization of the protection level, it should be noted that the grid of protection alternatives in Fig. 8.3 represents for each of the polders a continuous increase in safety along the two axes.

¹: dike section lengths are not drawn according to the scale, the same to Fig. 8.28 & 29.
8.4. Dike overflowing simulation

As discussed in Chapter 4, a set of simulations of the flow pattern in the transition zone, including both the flow in the river channels and into the polders, will generate the necessary information for decision analysis. The simulations should consider different boundary conditions, different river-dike configurations and failure mechanisms. The conditions for those simulations in the application are described below.

8.4.1. Overflow but no breaching

In the Netherlands, when a discharge at Lobith is used for hydraulic calculation in this area, a common practice by Rijkswaterstaat is to use the average discharge at Lith (the Meuse) corresponding with the given discharge at Lobith. Fully introducing the boundary condition of the Meuse, in the form of a discharge probability curve in the process of determining the probability curve of the overflow volume in polders, requires much more simulations and might have some technical problems (refer to Chapter 5). In this study the practice of using the average discharge at the Meuse in correspondence with a given discharge at Lobith is also adopted.

The design discharge wave of a peak value 15000 m$^3$/s at Lobith and design sea level of peak values 2, 3, 4, 5, 6 m (NAP) at Hoek van Holland, used by Rijkswaterstaat, are presented in Fig. 8.6 and Fig. 8.7. The design discharge waves for peak values other than 15000 m$^3$/s have been adapted from the flood wave in Fig. 8.6 proportionally to the flood peak values.

![Fig. 8.6. The design $Q_{\text{Lob}}(t)$ for 15000 m$^3$/s](image)

![Fig. 8.7. The design $H_{\text{HVH}}(t)$](image)

The recorded maximum discharge at Lobith and the sea level at Hoek van Holland is,
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respectively, 12280 m$^3$/s (occurred on 4 January 1926) and 3.85 m (occurred on 1 February 1953).

The peak value range of, respectively, the discharge at Lobith from $Q_{Lob}=3300$ to 16500 m$^3$/s and the sea level at Hoek van Holland from $H_{HVH}=2$ to 6 m are taken, resulting in 45 discrete boundary condition combinations (Table 8.1).

Table 8.1. Discrete boundary condition combinations

<table>
<thead>
<tr>
<th>$Q_{Lob}$</th>
<th>3300</th>
<th>4950</th>
<th>6600</th>
<th>8250</th>
<th>9900</th>
<th>11550</th>
<th>13200</th>
<th>14850</th>
<th>16500</th>
</tr>
</thead>
<tbody>
<tr>
<td>$H_{HVH}$</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Dike overflowing simulations are conducted to calculate the possible maximum overflow volume in polders under various boundary condition combinations for 16 alternatives and the do-nothing option (see the method and procedure described in Chapter 5). Figure 8.8 gives an example of the overflow volume in Polder 22 under the "do-nothing" alternative, in which the overflow volume caused under a boundary condition combination is indicated above the cross mark which represents the boundary condition combination.

![Overflow volume in Polder 22 for the existing dike heights](image)

Fig. 8.8. Overflow volume in Polder 22 for the existing dike heights

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With the calculated maximum overflow volume in polder $j$ ($W_j$), the contour lines of water volume in the polder are established for each polder and for each dike heightening alternative. For the example of Fig. 8.8, the contour lines in Polder 22 ($W_{22}$) can be presented in Fig. 8.9.

![Contour lines of water volume in Polder 22](image)

**Fig. 8.9.** Contour lines of water volume ($m^3$) in Polder 22 for the existing dike heights

Obviously, the sea level at Hoek van Holland has more influence on the overflow volume entering to Polder 22 than the discharge at Lobith (refer to Fig. 5.1).

### 8.4.2. Overflow and breaching

When the water level exceeds the dike height, dike breaching may occur. The ultimate inundation levels in the polder can be assumed to equate the lowest dike height of the polder (Boertien I). The inundation depth at a specific location in a polder is the difference between the inundation level and the altitude of the location. Example computations readily show that it would take in the order of a few days to completely fill a polder. Therefore, given the lowest dike heights of polders, the ultimate inundation level in each polder can be determined without any dike overflowing simulation. The overflowing situation at the existing dike height in Polder 22 is presented in Fig. 8.10.
8.5. Estimation of the exceedance probability of overflow volume

In the Netherlands, high storm surges occur mainly during the winter period (October to March). There are about 180 days per winter. The probability distribution of the sea level such as at Hoek van Holland is usually represented as the mean frequency of exceedance in high tides per year—$E(H_{HoH})$, whereas the discharge at Lobith is usually represented as the mean number of flood waves per year during which a certain discharge is exceeded—$E(Q_{Lob})$. Let $D(Q_{Lob})$ represent the average number of days per year during which that discharge is exceeded. $D(Q_{Lob})/E(Q_{Lob})$ represents the mean duration that a given discharge is exceeded by a flood wave.

The occurrence probability of hydrologic variables in the transition zone depends not only on the probability distribution at boundaries, but also on the degree of correlation between them. As described in Chapter 5, Van der Made (1969) concluded that there is no correlation between the sea level at Hoek van Holland and the discharge at Lobith.

Then Equation (5.3) can be operationalized into Equation 8.1.
\[
P(W_j) = \int \int_{\Omega} \left( p(Q_{\text{Lob}}) \times \frac{p(H_{\text{HvH}})}{180} \right) dQ_{\text{Lob}} dH_{\text{HvH}}
\]  

(8.1)

in which: \(P(W_j)\) is the probability of exceedance of \(W_j\); \(p(Q_{\text{Lob}})\) represents the density function of the \(D(Q_{\text{Lob}})\) curve (days per years); and \(p(H_{\text{HvH}})\) represents the density function of the mean frequency of exceedance in high tides per year, \(\Omega\) is the complementary area enclosed by the contour line of a particular value of \(W_j\) (see Fig. 5.7).

In order to conduct the calculation with Equation 8.1, the probability distributions or the numerical probability curves at boundaries are needed. In this section, the working line for the sea level at Hoek van Holland (a numerical exceedance probability curve) is used. For the discharge at Lobith, the working line (a numerical exceedance probability curve) adapted according to the three quantiles of discharge and their exceedance frequencies is used as recommended by Boertien I (MTPW, 1993).

The exceedance probability curve of \(W_j\) (the overflow volume) in each polder with each dike heightening alternative is integrated. With the topographic characteristics of polders, the inundation depths in each polder are derived from \(W_j\), and their exceedance probabilities are assumed to be equal to those of \(W_j\).

8.6. Flood damage assessment

8.6.1. Supporting information

Information on land use
As mentioned in Chapter 6, flood damage assessment requires several kinds of information, of which the property inventory in polders, flood loss values to different kind of properties, and the physical characteristics of the potential flood are most essential.

It is very difficult, if not impossible, for this study to make a property inventory of the four polders without the newly developed digitized land use map. In this application the ARC/INFO GIS is applied to manipulate the digitized map.

There are two basic descriptive information elements in ARC/INFO which are directly
related to the usage of GIS in this application: arc and polygon. Land use information can be grouped into arcs and polygons according to their spatial features. An arc is a string of \( \{x, y\} \) coordinates and thus a line feature. A polygon is an area defined by arcs that make up its boundary.

For flood damage assessment, land use information of an arc type can be a road, railway and other line infrastructure; land use information of a polygon type can be a piece of land for a certain purpose of use. The digitized land use information obtained from CBS is the digitized map, in which each piece of land use is classified into one of the 35 land use categories (see Appendix 8.1), and is drawn as a polygon. For the flood damage assessment of line infrastructures, e.g. road and railway, the length of the road, instead of the surface area of the road polygon, is more relevant. Obviously, if a road is digitized as an arc, its length is easily measured with ARC/INFO. Therefore, line infrastructure is digitized as an arc in the application.

The digitized land use map is merged with the dike and municipal borders to identify the information of arc and polygon, and indicate in which polder and municipality the arc or polygon are located. The average altitude (ground level) of a polygon is estimated on the basis of the topographic map.

**Information on flood loss values**

The depth-loss relationship for each land use category defined by CBS was established and updated to the 1994 price level (Zhou and Van der Heijden, 1994–c). These depth-loss functions were based on a Dutch report (Vrouwenmelder and Wubs, 1992-a). An English source (N’Jai, et al., 1990) was used to deduce the loss function of some land use categories when they are not present in the Dutch report. These functions have been applied in this application.

**Other socioeconomic information**

The indirect flood loss is assessed by the Value Added method (see Chapter 6). The assessment needs the following information:

- for each individual plant, such as factory and office: the activities, the number of employees and the location; and
- for each societal sector: the value added at the regional, or at least, national level.

Relevant information of industrial and service plants in the application area was
collected from the local Chamber of Commerce. There are 425 plants in the application area.

Information about the production value and value added for each economic sector is derived from CBS (1991).

By flood routing simulation, the duration of overflowing can be calculated. After flooding, the time needed to restore the industrial and service activities depends on many factors—for example, the severity of the flood, the capacity of the pumping facilities and the drainage system. It is impossible to predict the real inundation duration in the flood alleviation studies. Without the information on flood duration, as a simplification, 5 working days are arbitrarily assumed as the time needed for flood restoration in this application.

8.6.2. Annual expected flood damage

The annual expected damage differs with the extreme of overflowing. The annual expected damages of the two extremes form the interval of the annual expected flood damage.

**Overflowing only**

With the information described above, the tangible direct and indirect flood damage are assessed for each dike heightening alternative, respectively, with the unit-loss method and the value added method (see Chapter 6). In the literature, a maximum ratio of 1:1 between the tangible direct damage and intangible damage for residence is found (see Chapter 6). Lacking information on the intangible damage, 50% of the tangible direct flood damage for residence is used to roughly represent the intangible damage in this section.

Figure 8.11 presents the annual expected damage in the application area where the numerical probability curves of the discharge at Lobith and the sea level at Hoek van Holland are used. In this figure, Alt-0 represents the do-nothing alternative. The data are given in Appendix 8.2.
The calculated annual expected flood damage in different polders with different dike heightening alternatives illustrates the hydraulic correlation among river sections around the four neighbouring polders. For example, Alt-1 has higher dikes only in Polder 24 and Polder 22 in comparison with the do-nothing alternative. Dikes in Alt-1 are much higher in Polder 24 than those in Polder 22 in comparison with do-nothing, therefore annual expected damage in Alt-1 is much less than that in the do-nothing alternative in Polder 24 and is less than in Polder 22. Due to the hydraulic correlation, more water overflows into Polder 23 and Polder 21 in Alt-1, consequently, annual expected damage in Polder 23 and Polder 21 in Alt-1 is even larger than in the do-nothing alternative (Fig. 8.12).

Overflowing and breaching
Under the assumption that the ultimate inundation level in a polder equates the lowest dike height of the polder, the annual tangible and intangible flood damages are assessed. The expected annual flood damages are presented in Fig. 8.13. The data are given in Appendix 8.2.
Fig. 8.12. A comparison of annual expected damage for individual polders

Fig. 8.13. Annual expected damages (numerical, with breaching)

The flood damage in Polder 21 has been reported by Vrouwenvelder and Wubs (1992-a), who made a property inventory of Polder 21 for the study of the protection level for this...
polder. The flood damage assessed in this application with the aid of ARC/INFO is quite close to the flood damage reported by Vrouwenvelder and Wubs (1992-a). This gives confidence in the assessment of flood damage in this application.

8.6.3. Total expected flood damage reduction

For the optimization of alternatives, the present value of the expected flood damage reduction and the cost should be calculated. The value of the total expected flood damage reduction is obtained by projecting the annual expected flood damage reduction over the full service life and then calculating the present value of this time stream of damages at the base year.

In this application, 1994 is regarded as the base year, and it is assumed that all alternatives of dike heightening would be implemented at the base year (for the determination of the optimal protection level, see Chapter 4). A discount rate of 5% is used and the service life of a dike is taken at 50 years.

8.7. Estimation of dike heightening cost

Dike heightening involves cost of design, construction and maintenance. The design cost is relatively low in comparison with construction cost, and heightening the existing dike by the order of half meter (average heightening) will unlikely result in significant changes in the maintenance cost, therefore only the construction cost is considered in this application. The construction cost of dike heightening is calculated on the basis of the information of Zuidweg, et al. (1991). In this source, the costs for dike heightening vary in situations where there are, respectively, no buildings on dikes, buildings on one side, or on two sides. Data are provided for the Province South-Holland and the Province North-Brabant separately. The reported cost was based on the estimation and revised with the completed dike improvement projects. It is a linear function of the extra height which is added.

If there are buildings on dikes, the cost involved to replace the buildings is also considered as a part of the dike heightening cost. According to Zuidweg, et al. (1991), if there are buildings on one side and both sides of dikes, the cost is increased, respectively, by a factor of four and seven.
Polder 24 and Polder 23 are located in the Province of North-Brabant, and Polder 22 and Polder 21 in the Province of South-Holland. Without field investigation of the buildings on dikes, in the calculation, it is assumed there are buildings on both sides of the dike for those parts of a dike section which are located in the build-up area. Possible changes in the cost over the last years, with respect to the cost reported in Zuidweg, et al. (1991), were not considered in this application.

The costs for each dike heightening alternative are presented in Fig. 8.14. Figure 8.15 illustrates the protection cost for the grid of alternatives in function of the boundary conditions. The data are given in Appendix 8.2.

![Graph](image)

**Fig. 8.14. Cost for each dike heightening alternative (2-D)**

### 8.8. Evaluation of uncertainty

#### 8.8.1. Overview of uncertainty components

As indicated in Chapter 7, various kinds of uncertainties are involved in the decision analysis for dike heightening. In this section, four kinds of uncertainty are elaborated, the influence of the uncertainties on the expected flood damage reduction and further on the net benefits is analyzed.

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Fig. 8.15. Cost for each dike heightening alternative (3-D)

In Fig. 8.16, the four types of uncertainty considered in this application are illustrated (the shadowed blocks) (compare with Fig. 7.1).

To examine the influence of uncertainty on the decision analysis, the individual uncertainties are examined first followed by composition of overall uncertainty.

8.8.2. Uncertainty in the dike failure mechanism

The present value of the total flood damage reduction over 50 years, the present value of the cost of dike heightening, and the net benefit of each alternative are calculated for the two extremes of overflowing: overflowing only, and overflow and breaching. The numerical probability curves have been used to extrapolate the discharge at Lobith and the sea level at Hoek van Holland for the derivation of expected damage.

Figure 8.17 presents the response surface of dike heightening in the overflowing only case. For the extreme of overflowing only, Alt-01 has the largest net benefit and is identified as the optimal alternative for dike heightening.
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Fig. 8.16. Uncertainties considered in the application

using numerical probability curves

Fig. 8.17. Response surface of net benefit (no breaching)

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The volume of flood water entering into the polders is relatively small causing relatively low damage and thus also causing a low reduction of damage. The cost for protection on the other hand increase steeply with increasing levels of protection (see Fig. 8.14). This results in negative net benefits for most of the alternatives except in Alternatives 1, 2, 3, 4, 6.

The net benefit response surface for the overflow and breaching situation is presented in Fig. 8.18. For this extreme, for all alternatives, except Alt-6 and Alt-12, the expected flood damage reduction is much larger than the cost, and thus substantial net benefit is expected. For the breaching case the benefits strongly dominate the costs and the best solution is practically determined by the size of the benefits. Alt-11 represents the largest net benefits.

![Net benefit response surface](image)

**Fig. 8.18. Response surface net benefit (breaching)**

The response surface indicates a strong increase in the net benefits of protection from increasing peak discharge at Lobith. For the Alt-6 and Alt-12 the net benefits are very small, this corresponds to the following combined situations:

- the existing dike system is sufficient to protect from the low peak discharge at Lobith associated with these alternatives (thus no damage reduction);
- the high sea level has only limited influence on the polders in the middle and
upper part of the transition zone; and
- the most downstream polder(s) has a relatively high existing protection level, therefore the proposed upgrading has little effect on damage reduction.

8.8.3. Hydrologic uncertainty

There are 94 years' data of the discharge at Lobith (1901-1994) and 106 years' data of the sea level at Hoek van Holland (1889-1994). As indicated in Table 2.2, the working line of, respectively, the sea level at Hoek van Holland and at Lobith, with the modification of some quantiles at Lobith can still be used. Many types of parametric distribution have been used for the extrapolation of the boundary conditions. For example the Exponential distribution was recommended by the Delta Committee and the Becht Commission. A Pareto distribution was recently recommended as a parametric probability distribution for the sea level at Hook van Holland and the discharge at Lobith (Philippart, et al., 1994; Wijbenga, et al., 1993). The year maxima at Lobith and Hoek van Holland are fitted in this application with Gumbel I, Exponential and Log-Normal distribution to analyze the influence of using different probability distributions for the boundary condition extrapolation on the expected flood damage, in addition to the working line and Pareto.

When the Gumbel I, Exponential, Log-Normal distributions, the numerical probability curves and Pareto distribution are drawn in the same coordinate system, respectively, for the discharge at Lobith and the sea level at Hoek van Holland, it was found that a Pareto curve and Exponential curve well encompass the numerical probability curves. Therefore, this application takes the numerical, Pareto and Exponential distributions as the possible distributions to analyze their influence on the expected flood damage.

In Fig. 8.19 and Fig. 8.20, the numerical probability curve as well as the Pareto and Exponential distributions are drawn at several quantiles, respectively, for the discharge at Lobith and the sea level at Hoek van Holland.

If different probability distributions are, respectively, applied for the discharge at Lobith and the sea level at Hoek van Holland, \(3^2\) possible combinations of the probability distributions can be formed. In the present uncertainty analysis, a simplifying assumption is made to reduce the amount of computation: the same distribution type is used for both the discharge at Lobith and the sea level at Hoek van Holland. Thus three possible extrapolations are considered at each boundary.
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Fig. 8.19 & 8.20. Exceedance probabilities at a few $Q_{\text{Lob}}$ and $H_{\text{HWW}}$ with different distributions

When Pareto and Exponential distributions are applied for the extrapolation of the discharge at Lobith and the sea level for Hoek van Holland, the total expected flood damage reduction for the two extremes of overflowing are recalculated. The intervals of the total flood damage reduction and the cost for each alternative are presented in Fig. 8.21 and Fig. 8.22, respectively, for the two extremes, in which the mean is taken as the middle of the uncertainty range.

Fig. 8.21. Benefit interval due to hydrologic uncertainty (no breaching)
Fig. 8.22. Benefit interval due to hydrologic uncertainty (breaching)

It can be observed, in Fig. 8.19, that for the discharge at Lobith, the mean of the exceedance probability of Exponential and Pareto distribution, in particular in the upper range, is above the numerical curve. However, this is not the case with the sea level at Hoek van Holland (see Fig. 8.20). Therefore, considering the hydrologic uncertainty implies that a larger discharge is expected after the consideration for the same exceedance probability. The protection level against the discharge therefore should be upgraded. Hydrologic uncertainty can be expected to influence the ranking of alternatives.

8.8.4. Uncertainty in flood damage assessment

The standard flood depth-loss functions prepared and updated in Zhou and Van der Heijden (1994-c) have been applied in a study for the Municipality Roermond of the Province Limburg in the Netherlands (Zhou and Sanders, 1994). One of the results was that the total estimated tangible direct damage of a flood with the exceedance frequency 1/200 is 41 million Dutch Guilders. The Boertien II independently reported a quite close estimate of the flood damage based on other information sources of flood loss values (37.4 million) (MTPW, 1994). This indicates a consistent depth-loss function and gives
Considerable uncertainty remains however in the estimation of the flood damage. An impression of the magnitude of this uncertainty can be obtained from N’Jai et al. (1990). In this report, the mean flood loss value and standard deviation for some societal sectors—for example leisure, office, public buildings—at different inundation depths are presented. For the most of the sectors and at most inundation depths, the standard deviation is larger than the mean value. The largest standard deviation is about 8 times larger than the mean, with a sample size of 143.

Uncertainty on the estimation of tangible flood damage, due to an imperfect assessment method and incomplete/imperfect supporting information, has been assumed at ±50% of the estimated amount.

A maximum ratio of 1:1 between the tangible direct damage and the intangible damage for residence is reported in the literature (e.g. Green and Penning-Rosswel, 1989). In the previous analysis, 50% of the tangible direct flood damage for residence has been used to roughly represent the intangible effect. In this uncertainty analysis, 0% and 100% of the tangible direct flood damage for residence are regarded, respectively, to be the minimum and the maximum intangible damage.

Figure 8.23 and Fig. 8.24 present the uncertainty in the total expected flood damage reduction due to the uncertainty in the damage estimation for the two extremes of dike failure, when numerical probability curves are used.

8.8.5. Uncertainty in cost estimation

Uncertainty exists in the estimated cost of dike heightening. The uncertainty in the estimated cost of dike heightening has been assumed, similar to the assessment of flood damage, to have a range of ±50% of the estimated cost in this study.

The uncertainty interval of the expected flood damage reduction and the uncertainty interval of the cost are presented in Fig. 8.25 and Fig. 8.26, respectively, for the two extremes of dike failure.
Fig. 8.23. Benefit interval due to uncertainty in damage assessment (no breaching)

Fig. 8.24. Benefit interval due to uncertainty in damage assessment (breaching)
Fig. 8.25. Uncertainty interval of the benefit and cost (no breaching)

Fig. 8.26. Uncertainty interval of the benefit and cost (breaching)
8.8.6. Overall uncertainty

The dike failure mechanism has a dominant influence on uncertainty. The overall uncertainty intervals resulted from all individual uncertainty sources except the failure mechanism have been presented in Fig. 8.25 and 8.26. The upper and lower bands of the overall uncertainty contributed from all individual sources, including the dike failure mechanism, are presented in Fig. 8.27.

Given the uncertainty intervals of the total expected damage reduction and the cost of dike heightening, the uncertainty range for the net benefits can be determined as follows:

"upper range of net benefit = upper range damage reduction - lower range of cost"
"lower range of net benefit = lower range damage reduction - upper range of cost"

The uncertainty intervals of the net benefits for different dike heightening alternatives are presented in Fig. 8.27.

![Graph showing uncertainty intervals and mean of net benefits](image)

**Fig. 8.27. Uncertainty interval and mean of the net benefit**

The following observations can be made:

- the total range of uncertainty on damage reduction is most strongly influenced by
the uncertainty on the dike failure mechanism: the two extreme mechanisms enveloping this uncertainty have a widely varying impact on damage reduction; and
- the uncertainty interval on the benefits (damage reduction) is much larger than the uncertainty interval on the cost.

Alt-11 has the largest mean of the net benefit (see the arrow in Fig. 8.27): about 50 billion Guilders. The huge net benefit depends strongly on the assumption that the ultimate inundation level reaches up to the lowest dike height for the overflowing and breaching and the fact that a lot of dwellings and other properties are located in this application area.

8.9. Optimal Protection level for the combined system of polders

From Fig. 8.27, it can be observed that Alt-11 has the largest expected net benefits and is therefore indicated as the optimal solution. The very wide range of uncertainty however requires us to look more closely at the alternative solutions and the way they differ from Alt-11.

The dike failure mechanism has a dominating effect on uncertainty and two widely different solutions, namely, Alt-01 and Alt-11, emerge when the separate failure mechanisms are considered. More information is obviously required to bridge the large uncertainty gap and to merge towards on acceptable overall optimal solution.

In the absence of such information at the present stage, a satisfactory practical solution can be derived as follows:

It can be observed that the response surface of the expected net benefit in the breaching case (Fig. 8.18), which dominates the solution, demonstrates a steep gradient (increase in net benefits) for peak flow protection in the range of 8250 to 11550 m³/s and further a relatively low gradient for further increasing protection associated with increasing peak discharge at Lobith. As shown in Fig. 8.15, the cost for protection increase rather steeply for protection from higher Lobith peak discharges. Assessment of the influence of uncertainties in the previous sections has indicated a very large uncertainty of the benefits, which results from an accumulation of various uncertainties, and in comparison a modest uncertainty of the cost for protection.
The observation can then further be made that increasing benefits with a very large uncertainty have to be traded off against a relatively certain steep increase in the cost. Therefore a satisfactory (practical) solution to the flood protection problem, taking into account the present information, may be to select Alt-02 which reaps most of the potential benefits and avoids the steeply increasing cost for protection. The profile of the dike upgrading works, from downstream in upstream direction, associated with the Alt-02 and Alt-11 and compared to the existing situation, is presented in Fig. 8.28.

![Dike height diagram](image)

**Fig. 8.28. Some Dike heights before and after dike heightening**

The following observations can be made:

- Alt-02 results in upgrading those dike sections which in the past have not reached the same "average" protection level as the other sections;
- dike sections close to the sea boundary require little or no upgrading (for the profiled sections in Fig. 8.28 no upgrading is required, however, a slight upgrading is required for sections following a different profile path); this indicates (confirms) that protection of those sections has received more attention in the past; and
- Alt-11 increases substantially all dike sections in the middle and upper part of the transition zone.
Application

In summary: selection of Alt-02 focuses on upgrading of the weak spots in the existing protection system and in doing so avoids sharply increasing costs for upgrading for which the associated benefits are highly uncertain.

8.10. Comparison with earlier proposed protection levels

In earlier documents (Delta Committee, 1962; MTPW, 1977; MTPW 1993) particular protection levels were proposed for the dike systems which were uniformly applied to all defense systems in a particular geographical zone. An equal protection level for the individual systems in a zone may be preferred by politicians. As has been reasoned in the introductory chapters a substantial basic protection level is already provided. Concerning the further upgrading, there is a growing awareness for a differentiated protection in relation to the value to be protected and the objective of optimizing the use of scarce resources. In practice all 53 polder systems in the Netherlands have a different protection level and the concept of different protection levels is thus not new. The discussion focuses then on the setting of targets for further upgrading.

Initially (Delta Committee, 1962) a protection level of 1/10000 and 1/4000 were proposed respectively for the coastal protection and the inland protection with no differentiation between the river zone and transition zone. In the later Boertien I study, based on more detailed evaluations, a protection level of 1/1250 was proposed and adopted for the river zone. No further specifications, based on more detailed elaborations, have been made yet for the transition zone.

Figure 8.29 illustrates the existing dike height in comparison with the 2000 years flood line (Koster, 1990), and the optimal dike height derived in the application in this chapter. The coastal protection level with a return period of 10000 years is obviously above the 2000 years protection line; in the transition zone the existing and presently indicated protection levels are substantially higher than the 2000 years protection. The heights become more comparable in the upper part of the transition zone.

In this thesis the dike height is determined based on the net benefits and this results in dike heights well above the 2000 years line and in the range between 2000 and 10000 years. This corresponds to the high value to be protected in this populated and economically very active part of the Netherlands. This is particularly so if the most drastic flooding mechanism, based on breaching, is considered.
Fig. 8.29. Some dike heights in this application and given by Rijkswaterstaat

8.11. Overall optimum protection level for the polders

In the previous section Alt-11 was selected as the best alternative based on the mean of the expected net benefit. Figure 8.30 illustrates the net benefit response surface associated with this choice.

The optimal protection alternative was established for a particular combination of the boundary conditions which is kept the same for each polder. The condition that the boundary condition is the same for each polder should now be relaxed in order to potentially further improve the total flood control performance (maximization of overall net benefits).

The individual response surfaces for the four polders for the 16 alternatives are presented in Fig. 8.31 to Fig. 8.34. It can be observed that each of them has a different response surface with a different maximum net benefit.
Application

Fig. 8.30. Response surface of dike heightening for the combined polder system

Fig. 8.31 and Fig. 8.32. Response surfaces of dike heightening for Polder 21 and 22

Fig. 8.33 and Fig. 8.34. Response surfaces of dike heightening for Polder 23 and 24
Chapter 8

It should be realized that each point of the response surface corresponds to a particular combination of measures which also determines the physical system. It can however be reasonably assumed that the collection of points gives a good representation of the shape of the response surface.

The gradient search prepared in Chapter 4 aims at improving the overall net benefits from flood control by testing changes in the protection level for the individual polders with the combined (restricted) solution as a starting point.

For this particular case this means, in first instance, testing the different possibilities for the level associated with a reduction in the sea level boundary from 5 to 4 m (Alt-05) or a reduction of the upstream boundary peak level from 16500 to 14850 m$^3$/s (Alt-10).

Theoretically all the possible changes for the four polders from Alt-11 to 10 or from Alt-11 to Alt-05 should be tested to incorporate changes for each of the individual polders as well as combinations of such changes. For each of the possible changes, the new combination should be tested as a new alternative and analyzed as the other alternative. The total expected benefit should then be compared with the optimal point (Alt-11).

In order to limit the computational burden an approximate gradient search has been carried out. As long as the changes from the base point are not too large then the gradient can be approximated quite well by the gradients on the response surfaces indicated in e.g. Fig. 8.33. Limiting further the test to the individual changes for each polder, the following result was obtained:

- For polder 21, 22, 24, the gradients to change from protection level $M_{21}(16500,5)$, $M_{22}(16500,5)$ and $M_{24}(16500,5)$ are negative.
- For polder 23 the gradient is positive for changes from $M_{23}(16500,5)$ to $M_{23}(16500,4)$, $M_{23}(14850,4)$, $M_{23}(13200,4)$, $M_{23}(13200,5)$, $M_{23}(11550,5)$ and $M_{23}(11550,4)$.
- The change to $M_{23}(11550,4)$ results in the largest positive gradient of net benefit: a cost reduction or an increase of net benefit of 7.45 (million Fl).

Since Polder 23 has less property of high value, the flood damage reduction for this polder is relatively low in comparison with other polders, thus upgrading protection to a high level is not cost effective.
The optimal overall protection level maximizing net benefit for the total system consists then of implementing the measure set $M_{21}(16500,5)$, $M_{23}(11550,4)$, $M_{22}(16500,5)$ and $M_{24}(16500,5)$.

Knowing the proximate solution with this relatively course discreteness of the boundary conditions a more detailed analysis could be made around the optimal solution using e.g. 10 cm intervals for the level at Hoek van Holland and steps of 500 m³/s at Lobith.

8.12. Conclusions

In this chapter, 16 protection levels, which can protect the application area as a whole from floods covering all possible severity ranges, have been proposed. In the first round of decision analysis, dike heightening alternatives with the same protection level for all polders have been formulated. The expected flood damage reduction and the cost of dike heightening have been calculated for each alternative. The influence of four kinds of uncertainty has been analyzed.

Some of the proposed alternatives have a relative better protection against large river discharges than high sea levels such as Alt-05 and Alt-11; some are the opposite, such as Alt-06 and Alt-12. Overflowing under large discharge is more likely to induce breaching than overflowing under high sea level if the water levels in front of the dike are at the same order, since the duration of the discharge wave is much longer than the sea level. The difference in the likelihood of breaching has not been considered in this application due to the difficulty in quantifying the likelihood. However, knowing the maximum recorded discharge at Lobith and the sea level at Hoek van Holland, it can be stated that Alt-12 and Alt-06 have a relatively low protection against large discharges and therefore are less promising than other alternatives.

Alt-11 has been found to be the economically optimal alternative among the 16 ones under the following assumptions: (1) the adverse social and environmental impact of dike heightening is excluded; (2) both the extreme of overflowing with and without breaching should be considered; (3) the numerical probability curves, the Pareto and the Exponential probability distribution can represent the range of the probability distribution for the extrapolation of the discharge at Lobith and the sea level at Hoek van Holland; (4) the estimated tangible flood damage has an uncertainty range of ±50%; (5) the intangible flood damage has an uncertainty range from 0% to 100% of the tangible
direct flood damage for housing; (6) the estimated cost of dike heightening has an uncertainty range of ±50%.

The four kinds of uncertainty have different influences on the decision on dike heightening. In the following order of decreasing importance: (1) the uncertainty in overflowing mechanism, (2) the hydrologic uncertainty associated with the use of different probability distributions for the boundary condition extrapolation, and (3) the uncertainty in the estimation of cost for dike heightening and in the assessment of flood damage.

The economically optimal alternative of dike heightening can protect the application area from flood caused by overflowing and breaching under the discharge of 16500 m³/s at Lobith and the sea level of 5.0 m at Hoek van Holland. Based on the response surface of dike heightening of each individual polder, this overall protection level has been further differentiated in Polder 23 to the protection level corresponding to the discharge of 11500 m³/s and the sea level of 4 m at Hoek van Holland.

Given the very large uncertainty involved in the damage reduction, Alt-02 has been selected, which can protect the whole area from flooding under the combination of the discharge of 11500 m³/s at Lobith and the sea level of 4 meter at Hoek van Holland. This selected combination focuses on upgrading of the weak spots in the existing protection system and avoiding sharply increasing upgrading costs with highly uncertain benefits.
Appendix 8.1. Land Use Categories Defined by CBS

1 railways, tramway and metros
2 improved roads (including verges)
3 other roads
4 water reservoirs
5 other water wider than 6 meter
6 cemeteries
7 sports ground
8 airfields and airports
9 allotments
10 dumping sites
11 car wreck sites
12 mining area
13 parks and public gardens
14 holiday recreations
15 recreational objects and area
16 social-cultural facilities (hospitals, education, club, buildings, museums etc.)
17 other public facilities (utility services, storage yards for public authorities)
18 industrial areas (including car parks, offices and other auxiliary buildings)
19 water with a main function of recreation
20 other trade area (e.g. shops, offices, banks, hotel and catering)
21 residential areas
22 buildings sits for industrial and harbour areas
23 buildings sites for other purposes
24 woodland
25 glasshouses (only for agriculture)
26 other agricultural use
27 dry natural area (dry heaths, dunes, and sandy beaches)
28 wet natural area (wet heaths, moors)
29 other areas

1: only the categories present in the application area are listed
Appendix 8.2. Annual expected flood damage and cost for dike heightening

Table 8.2.1. Annual expected tangible flood damage without breaching

<table>
<thead>
<tr>
<th>Alt</th>
<th>annual tangible damage (Fl.) ²</th>
<th>annual intangible damage (Fl.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>P-21³</td>
<td>P-22</td>
</tr>
<tr>
<td>0</td>
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<td>3217923</td>
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<tr>
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<td>178436</td>
<td>3217706</td>
</tr>
<tr>
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<td>178495</td>
<td>3214244</td>
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<tr>
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<td>178435</td>
<td>3214888</td>
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<td>07</td>
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<tr>
<td>16</td>
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<td>1431</td>
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</table>

1: alternative numbering, "0" represents do-nothing Alternative;
2: using numerical curves for the extrapolation; ³: Polder 21

This table needs further explanation. As mentioned above, there is a correlation among the hydrologic variables such as the river water level around the four neighbouring polders. If dikes are heightened only in one polder, the overflow volume and the expected flood damage in this polder will be less. However, due to the hydraulic correlation, the river water levels around other polders will likely rise and in turn increase the overflow volume and expected flood damage in these polders.
### Application

Table 8.2.2. Annual expected tangible and intangible flood damage (with breaching)

<table>
<thead>
<tr>
<th>Alt&lt;sup&gt;1&lt;/sup&gt;</th>
<th>tangible damage (FL)</th>
<th>intangible damage (FL)</th>
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</thead>
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</tr>
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<td>1.39E8</td>
</tr>
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<td>1.36E9</td>
<td>5.57E7</td>
</tr>
<tr>
<td>02</td>
<td>1.36E9</td>
<td>2.41E7</td>
</tr>
<tr>
<td>03</td>
<td>1.36E9</td>
<td>1.85E7</td>
</tr>
<tr>
<td>04</td>
<td>1.36E9</td>
<td>1.46E7</td>
</tr>
<tr>
<td>05</td>
<td>1.36E9</td>
<td>6.68E6</td>
</tr>
<tr>
<td>06</td>
<td>1.36E9</td>
<td>1.39E8</td>
</tr>
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<td>1.08E7</td>
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<td>8.79E6</td>
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<sup>1</sup>: alternative numbering, "0" represents do-nothing Alternative;

<sup>2</sup>: Polder 21

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Table 8.2.3. Cost of dike heightening (in million Guilders)

<table>
<thead>
<tr>
<th>Alter. No.</th>
<th>Polder 21</th>
<th>Polder 22</th>
<th>Polder 23</th>
<th>Polder 24</th>
<th>total cost</th>
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CHAPTER 9
SUMMARY AND CONCLUSIONS

9.1. Summary

In recent years the so-called Boertien I and II studies (MTPW, 1993 and 1994) have incorporated the latest insights and political and societal interests in the further upgrading of dike systems in the Netherlands. They form the latest in a long series of treatises on flood protection in the Netherlands. Environmental concerns associated with the further upgrading of the dikes have strongly entered into the evaluation. The large floods in the winter of 1994 and especially in 1995 have further drawn an increasing attention to adequate protection by the dikes. The unprecedented evacuation of 250000 persons and millions of livestock have demonstrated the feasibility of evacuation as a non-structural measure. Damage to property in flood prone areas has further intensified the discussion on the responsibility for damages incurred in areas with known risks and a reduction of the damage potential.

The present need to upgrade an existing elaborate flood defence system in the Netherlands as well as the necessity to alleviate the flood potential in other countries and the emergence of an increasingly wide range of alternative planning and design options in flood alleviation have increased the demand for a rational and quantified approach to risk assessment. The complexity in the methodology of risk assessment for flood alleviation calls for further contributions to a systematic and operational planning methodology. In particular for the complex situation in the transition zone a further operationalization is in order.

The overall objective of this study has been to formulate a framework for planning and design of flood alleviation in the transition zone, focusing on river dike improvement, and to operationalize and apply the approach. The transition zone has received less attention than flooding in the tidal and river zones. The interaction between up- and downstream boundary conditions and the interdependence of flood alleviation measures pose special challenges for risk assessment. The study has focused on dike heightening as a structural measure for flood alleviation. As mentioned above and elaborated in the first chapters of this study, river dike improvement forms only part of a larger set of
possible measures for flood alleviation.

Flood protection in the transition zone is characterized by interdependence of flood control measures and the joint influence of upstream and downstream boundary conditions. The flood alleviation problem for the transition zone has been treated as a decision problem with a high dimensionality and large uncertainty. Based on a systematic analysis of the different components it has been proposed to decompose the total problem into a number of sub-problems. The first step, determination of target protection levels for a given physical and socioeconomic system, is followed by steps to take care of the dynamics of the implementation of flood control works and long term adjustment to changes in the system. A method has been proposed to trace the response surface (net benefits) and identify the optimal solution, based on the net benefit of dike heightening, among a very large set of possibilities.

Operationalization has focused on three essential components of risk assessment in flood alleviation: estimation of flood magnitudes and occurrence probabilities, assessment of flood damages, and uncertainty analysis.

For floods induced by dike overflowing, the flood water volume in polders determines the inundation depth in polders. Methods and procedures for the estimation of flood water volume entering polders in the river transition zone and for the determination of the exceedance probability have been formulated. This involved calculation of unsteady dike overflow discharges from the potential flood events with a one-dimensional open channel flow model.

The assessment methods for flood damages presently available are either at a fine geographic level or only suitable for the assessment of a severe flood event. Accordingly, assessment methods are required for a flood alleviation study at the regional level or the level of a river basin, as well as for less severe flood events. A GIS-assisted land use inventory has been applied for the assessment of the tangible flood damage, a Value Added method has been developed for the assessment of tangible indirect damage; the intangible flood damage has been incorporated through the magnitude relationship of different kinds of flood damages.

A mapping of the different sources of uncertainty has been carried out, incorporating parameters related to the physical system as well as the damage and cost assessment. The influence of individual uncertainty components on the final result, net benefits, has
Summary and conclusions

been quantified in the application. In previous studies (e.g. Delta Committee, 1962; MTPW, 1977 and 1993) only the breaching case was considered. In this study the influence of the uncertainty on the dike failure mechanism has been incorporated by fully analyzing two failure mechanisms which delineate a minimum and maximum effect on flood damage, namely, dike overflow only and overflow followed by breaching.

9.2. Discussions and conclusions

Formulation and operationalization of a decision framework for risk assessment and the application, have led to the following observations and conclusions:

(1) The decision process is strongly characterized by a trade-off between very high but also very uncertain benefits versus relatively low costs for dike heightening with far less uncertainty.

(2) The uncertainty on the dike failure mechanism has the strongest influence on the total uncertainty of the performance of alternative protection levels. The optimal solution (maximum net benefit) for the two failure mechanisms varies widely, from a minimal upgrading of low sections in the existing dike sections, to a substantial heightening of most dike sections. More information on the failure mechanisms should allow to narrow the gap between the two solutions associated with two dike failure mechanisms and to determine more closely the true optimum. With the present information a very large uncertainty on damage reduction has to be traded off against a relatively certain sharp increase in cost for increased protection. The magnitude of the damage in case of breaching suggests that if breaching is indeed the dominant mode of failure for the Dutch dike systems, then specific alternatives which prevent or obstruct breaching, seem attractive protection measures in association with (or even replacing) dike heightening.

(3) In the application a response surface for flood protection has been established in function of the two boundary conditions. Tracing of this response surface based on the total system safety level, followed by a gradient search to establish the optimal protection level for each polder, appears an efficient way to identify the overall optimal solution out of a very large set of possible solutions.

(4) Determination of the joint probability function for the two boundaries remains a sensitive point in the risk assessment. The method followed by Rijkswaterstaat using one steady and unsteady boundary is an approximation which leads to an overestimation of the flood occurrence. This study has improved, with the aid of
Rijkswaterstaat, the current practice of using one steady and one unsteady boundary for flood simulation, by treating both boundary conditions as unsteady. More insight into the meteorological circumstances which create high flow at the upstream river boundary and high water level at sea level and into the possibility of coinciding circumstances should further improve the approach.

9.3. Limitations of the study

Considerations of feasibility have led to a range of self-imposed limitations in the scope and content of this study.
- The research has focused on river dike improvement in a system of polders in the transition zone rather than flood alleviation at the level of the entire river basin.
- Due to limitations of available data, full integration of structural and non-structural measures could not be achieved.
- A multiobjective decision model has been formulated, but flood damages on environmental assets and socio-environmental impacts of dike heightening could not been considered in the application.
- Two failure mechanisms associated with dike height have been considered: overflowing and breaching. Other failure mechanisms and design variables which have to assure the hydraulic and geotechnical stability of the dike have been assumed to be handled separately in design development.
- Possible long term changes in sea level, regional climate and upstream river basin development have not been incorporated in the estimation of boundary conditions.
- Investment planning and scheduling under budget constraints of river dike improvement have been addressed at the conceptual level and included in the formulation of a quasi-dynamic multiobjective decision model but could not be fully implemented in the application.

The following issues are recommended for further studies to overcome some of these limitations.

9.4. Recommendations

The preparation and application of a methodology for risk assessment in the transition zone have focused on the height of the dikes as a main parameter in the planning of
Summary and conclusions

protection from flooding. However as mentioned at several places in this study this forms only part of a wider range of possibilities for flood alleviation. The development and application of the present methodology has triggered some thoughts on a number of related subjects which fit into this wider scope, and which have caught particular attention as a result of the 1994 and 1995 floods in the Netherlands. The following subjects can be mentioned.

Integration of structural and non-structural flood alleviation measures

The ultimate goal in risk assessment is to find the most effective combination of structural and non-structural measures. In the framework for risk assessment elaborated in this study three main parts can be distinguished, namely, a) estimation of flood occurrence, b) determination flood volume and inundation processes; and c) damage assessment. Those three steps play an essential role in the evaluation of structural measures. Non-structural measures influence only the magnitude of the damage and relate therefore only to part c).

The damage evaluation method developed in this study may be further expanded in order to analyze the effects of non-structural measures. GIS may especially be used to efficiently include alternative non-structural measures. In practice the burden of preparation of such inventories and consistently analyzing alternative options has often prevented such comparison. Following the translation of non-structural measures into an appropriate damage pattern, the full framework can be used again to assess different alternative alleviation strategies consisting of combinations of structural and non-structural measures. In such way both types of measures may be integrated and evaluated on, for example, the multiobjective basis.

GIS assisted land use zoning

Digitization of land use in the Netherlands by the CBS is in progress and reaching completion. In the damage assessment carried out in this study, this land use map was used as a base map, and boundaries of primary (outer) and secondary (inner) dikes and line infrastructure were added to facilitate the preparation of inventories and damage assessment. Without the digitized land use map, it would have been practically impossible in the course of this study to make a property inventory of the polders. As mentioned above this GIS instrument can be further extended to assist in the evaluation of non-structural measures. Some useful extensions are the following:
If some highly valued properties receive extra protection—for example, by heightening the inner dikes of a polder—then the total expected flood damage of a polder may be reduced greatly. For this purpose, GIS-assisted models can identify those highly valued properties and analyze the effectiveness in reducing expected flood damage with some special structural measures.

There is a need to investigate different land use patterns associated with long term physical planning. The GIS processing of such options can be made more efficient by adding appropriate procedures for modifying land use patterns.

For the preparation of requirements for- and evaluation of the consequences of evacuation, a more detailed level of information is required than the CBS land use map currently provides. This information is often available at a local level. Overlays should be prepared which, used together with the base map, would provide the necessary information.

The results from the flow simulation model (inundation depth in different polders) could be linked with the GIS data base to present the consequences of alternative flood control measures for selected flood scenarios.

Influence of changes in the river basin

The large floods in two consecutive years 1994 and 1995 have drawn attention to the changes in the upstream part of the Rhine and Meuse basins and the possible consequences on the generation of larger floods for the downstream part of these basins. In the last decade an increasing urbanization has taken place as well as improvements to drainage. Such changes definitely change the runoff pattern, however due to its distributed nature, it is difficult to determine exactly the reference situation and the magnitude of the impact. Floods can originate from many different circumstances and only a few events of extreme floods are available to describe its population; large uncertainty is therefore associated with extrapolation.

Larger and more frequent flood waves at the upstream boundary require an adjustment of flood frequencies. New damage computations can be carried out with such a new probability density function, resulting in increased damage. It is then conceivable to determine the downstream increase in damage due to the upstream changes. It should then however be noted that such assessment carries an extra amount of uncertainty on top of the large uncertainty illustrated in this study; it would therefore be difficult to use such computations as a basis for compensation.
Summary and conclusions

An analogy can be observed between the changes in the river basin and their influence on flooding and the interdependence of flood control measures in the transition zone. In the transition zone the interactions originate from hydraulic interactions, the changes in runoff pattern can be called hydrologic interaction. There is however a basic difference: the interdependence in the transition zone leads to a large combinatorial decision problem while the changing run-off pattern "only" changes the boundary condition.

Alternative dike design

Lately alternative dike designs ("smart" designs) have been prepared basically in response to environmental concerns with respect to the proposed further dike heightening. Additional improvement of the stability of the existing and new dikes in the Netherlands is required. Such new designs have less adverse impact on the environment and will influence (improve) the overall dike stability and in particular its failure mechanisms. The widely different results for different failure mechanisms have illustrated that the economically optimal protection can be easily reached without substantial dike heightening if dikes are less sensitive to breaching, and with substantial dike heightening if dikes are sensitive to breaching.
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NOTATION

$A^y$: maximum limit on available implementation capacity in year $y$;

$B_1, B_2$: benefit of a project;

$B_j(l, y)$: use of budget in year $y$ by flood control project $j$ after it started in year $l$;

$B_e^j(P_1^j, P_0^j)$: present value of the stream of expected future economic benefits (damage reduction) for polder $j$ for which protection level is proposed to upgrade from $P_0^j$ to $P_1^j$;

$B_h^j(P_1^j, P_0^j)$: for the reduction of human life loss, similar to $B_e^j$;

$B_m^j(P_1^j, P_0^j)$: for the reduction of the environmental damage, similar to $B_e^j$;

$B_s^j(P_1^j, P_0^j)$: for the reduction of the social damage, similar to $B_e^j$;

$C_1, C_2$: cost of a project;

$C_e^j(P_1^j, P_0^j)$: expected value of the cost for implementation of dike heightening measure to upgrade to the protection level $P_1^j$;

$C_m^j(P_1^j, P_0^j)$: for the environmental impact, similar to $C_e^j$;

$C_s^j(P_1^j, P_0^j)$: for the social impact, similar to $C_e^j$;

$D(p, P)$: damage of a flood event with occurrence probability $p$ under the proposed protection level $P$;

$D(p, P_0)$: damage of a flood event with occurrence probability $p$ under the present protection level $P_0$;

$D(Q_{Lob})$: average number of days per year during which the discharge at Lobith is exceeded;

$E(H_{HvH})$: mean frequency of exceedance in high tides per year at Hoek van Holland;

$E(Q_{Lob})$: mean number of flood waves per year during which a certain discharge at Lobith is exceeded;

$E^y$: maximum limit on available budget in year $y$;

$F_1, F_2$: expected increase and decrease number of human life loss;

$H_d$: downstream water level;

$H_{HvH}$: sea level at Hoek van Holland;

$I_j(l, y)$: use of implementation capacity by flood control project $j$ in year $y$ after it started in year $l$;

$j$: index of a particular flood control project (measure);

$k$: the risk-aversion-parameter;

$K_s$: social criterion factor to distinguish structure types;

$l$: starting year index;
Notation

\(M_j(Q_{\text{Lob}}, H_{\text{HV}})\) : flood alleviation measure for polder \(j\) to protect this polder against the situation arising from the discharge at Lobith \(Q_{\text{Lob}}\) and the sea level at Hoek van Holland \(H_{\text{HV}}\);

\(n\) : the number of polders;

\(N_a\) : number of places where the activity under consideration is carried out;

\(n_d\) : design life of a project;

\(N_p\) : number of people at risk;

\(n_r\) : number of people at risk in the event of a failure;

\(p(H_{\text{HV}})\) : probability density function of the mean frequency of exceedance in high tides per year at Hoek van Holland;

\(p(Q_{\text{Lob}})\) : probability density function of the \(D(Q_{\text{Lob}})\) curve (days per years);

\(P(W_j)\) : probability of exceedance of \(W_j\);

\(P_0\) : present protection level;

\(P_0^j\) : the present protection level \(P_0\) (exceedance probability) for polder \(j\);

\(P_d|P_0\) : the probability of being killed in the event of an accident or an activity;

\(P_f\) : the probability of failure due to any cause during the design life \(n_d\) years;

\(P_i\) : proposed protection level;

\(P_i^j\) : the proposed protection level \(P_i\) for polder \(j\);

\(P_n(i)\) : the individually acceptable level of risk related to activity "i" per year;

\(P_s(i)\) : the socially acceptable level of risk related to activity "i" per year;

\(Q(t)\) : overflow discharge process at dike section \(i\);

\(Q_{\text{Lob}}\) : discharge at Lobith;

\(Q_u\) : upstream discharge;

\(r\) : discount rate;

\(R\) : resistance;

\(R_d\) : design resistance;

\(S\) : load;

\(S_d\) : design load;

\(T\) : life service of the flood protection system;

\(V_j^1\) : present value of the net benefit of project \(j\) when it starts in year \(1\);

\(X_j^t\) : 0 or 1 integer variable, if \(= 1\), then project \(j\) starts in year \(t\);

\(y\) : construction year index;

\(W_j\) : flood volume in polder \(j\);

\(\beta\) : the policy factor.
Naar een Operationele Aanpak van Overstromingsrisiko

Samenvatting

Hao-Ming ZHOU

1. Doelstelling van de studie

Deze studie werd opgezet ter ondersteuning van het beslissingsproces over het bepalen van het optimale protektie-niveau van polders door middel van dijken tegen overstroming. De aandacht wordt hierbij geconcentreerd op de dijkhoogte bepaling als de strategische ontwerp variabele.

Ongeveer de helft van Nederland bevindt zich beneden het gemiddelde zeeniveau en dient beschermd te worden tegen overstroming d.m.v. dijken en natuurlijke duinen. Het totale laaggelegen gebied is ingedeeld in 53 polders, elk beschermd door een individuele dijkring. Naar de aard van het overstromingsrisiko kan Nederland ingedeeld worden in drie zones nl:

- het rivierengebied, waar het overstromingsrisiko alleen bepaald wordt door de afvoer op de rivieren;
- het kustgebied, waar het overstromingsrisiko afhankelijk is van stormvloeden op zee; en
- een transitiegebied waar zowel hoge afvoer op de rivieren en hoge waterstand aan de kust van invloed zijn op het overstromingsrisiko.

Het beslissingsproces voor het transitiegebied heeft specifieke kenmerken. Deze studie richt zich speciaal op het beslissingsproces voor deze zone.

Beleid ten aanzien van dijkverhoging is in het laatste decennium aanzienlijk complexer geworden door een grotere maatschappelijke betrokkenheid en een groter aantal alternatieve opties. Beslissingen over dijkverhoging zijn verder gekenmerkt door een grote mate van onzekerheid over de uiteindelijke netto baten. Deze ontwikkeling vraagt om een methode om de alternatieve opties te kwantificeren en te vergelijken.

Deze studie richt zich op het conceptualiseren, operationaliseren en toepassen van een dergelijke methode voor het transitiegebied.
Samenvatting

2. Overzicht

Dit proefschrift is als volgt gestructureerd:
- Inleiding: historische overzicht van studies in Nederland en identifikatie van onderwerpen die verdere onderzoek behoeven; huidige gangbare benaderingen voor analyse van het overstromingsbeheer (hoofdstukken 2 en 3).
- Ontwikkeling van een raamwerk voor analyse van het overstromingsbeheer voor de transitiezone gericht op beslissingen over dijkhoogte; formulering van het optimalisatie probleem met inbegrip van de dynamische aspecten, meervoudige doelstellingen en onzekerheid; afleiding van een praktische oplossing voor het optimalisatie probleem (hoofdstuk 4).
- Het bepalen van een optimale beschermingsstrategie (streefniveaus) voor een gegeven fysische toestand van het stroomgebied en sociaal-economische situatie in het potentiële overstromingsgebied vormt het belangrijkste onderdeel van het analyseren van het totale beheersprobleem. Dit probleem kan opgesplitst worden in drie onderdelen. Voor elk deel werd een analyse module geoperationaliseerd.
- Deze drie modules zijn:
  - simulatie van het overstromingsvolume in functie van de grenswaardes en een bepaalde dijkverhogingsstrategie;
  - evaluatie van overstromingsschade met inbegrip van direkte, indirecte en niet-monetaire schade; gebruik van GIS bij de inventarisatie van grondgebruik; en
  - bepaling van een onzekerheidsmarge op de netto baten van een bepaalde dijkverhogingsstrategie.
- Gezamenlijk leveren de drie modules de informatie ten ondersteuning van het beslissingsproces (hoofdstukken 5, 6 en 7).
- De methodologie werd toegepast op vier polders in de transitie zone in Nederland (hoofdstuk 8).

3. Belangrijke karakteristieken van het beslisprobleem

Uit de analyse van het beslisprobleem komen met name de volgende elementen als belangrijk naar voren.
- hoge dimensionaaliteit van het optimalisatie probleem:
  Bescherming tegen overstroming in de transitiegebied kenmerkt zich door afhankelijkheid van de oplossingen van een aantal variabelen: verandering van de
Naar een operationele aanpak van overstromingsrisiko

dijkhoogte voor een bepaalde dijksektie heeft invloed op het overstromingsgedrag voor andere delen; verder dienen combinaties van de beneden- en bovenstroomse grenswaarden beschouwd te worden; een en ander leidt tot een groot aantal mogelijke combinaties die afgetast dienen te worden om de optimale strategie te bepalen.

- dynamische aspekten:
Het totale overstromingsbeheer omvat het bepalen van het optimale beschermingsniveau voor de individuele polders, rekening houdend met dynamische aspekten van implementatie en programmering van dijkverbeteringswerken en langere termijn aanpassing van de streefniveaus voor bescherming. De operationalisering van de methodologie in deze studie heeft zich beperkt tot het bepalen van optimale streefniveaus.

- onzekerheden:
Het beslissproces wordt gekenmerkt door een inherente onzekerheid ten aanzien van het optreden van extreme meteorologische/hydrologische omstandigheden en onzekerheden die samenhangen met het bepalen van het overstromingsproces resulterend in een bepaalde schade. Onzekerheid over het dijkbezwijkproces vormt hierin een belangrijke component.

4. Operationeel analyse instrument

Analyse van het overstromingsbeheer omvat het verzamelen en bewerken van een grote hoeveelheid gegevens, het beschouwen van een aantal complexe processen en het structureren van de afwegingen bij de keuze van de optimale strategie. Het analyse proces dient verder zo ingericht te worden dat een grote aantal alternatieven kan worden beschouwd. Operationalisering van het analyse proces is dan ook een voorwaarde om tot een goed overzicht van mogelijkheden te komen en de afwegingen konkreet te maken.

Operationalisering zoals uitgewerkt in deze studie heeft zich toegespitst op het formuleren van drie modules, gericht op:

- simulatie van het overstromingsproces,
- evaluatie van de overstromingsschade,
- bepaling van de onzekerheidsmarge.
Samenvatting

De struktuurering van het afwegingsproces omvat het aftasten van het netto baten responsievvlak in functie van alternatieve dijkverhogingsmaatregelen. Dit aftasten gebeurt in twee fasen, te weten:

- de bepaling van een globaal protektie-niveau voor de polders als geheel, gevolgd door
- een differentiering van het protektie-niveau voor de individuele polders.

5. Conclusies

Formulering en toepassing van het operationeel raamwerk ter ondersteuning van het overstromingsbeheer heeft geleid tot de volgende hoofdconclusies:

- de beslissing ten aanzien van dijkverhoging wordt in sterke mate gekarakteriseerd door de afweging van zeer grote doch erg onzekere baten ten opzichte van relatief beperkte doch praktisch zekere kosten voor een hoger beschermingsniveau;
- de onzekerheid over het dijkbezwijkmechanisme (volledig doorbraak ten opzichte van alleen stroming over de top) heeft de grootste invloed op de beslissing; dit leidt tot de conclusie dat wellicht meer aandacht dient geschonken te worden aan maatregelen die specifiek het volledige doorbreken van de dijken tegengaan;
- bij het zoekproces naar de optimale protektie niveaus werd het netto baten responsievvlak geformuleerd in functie van maatregelen geassocieerd met combinaties van boven- en benedenstroomse grenswaarden; het zoekproces kwam op deze manier snel tot de optimale beschermingsstrategie; en
- bepaling van de gezamenlijke waarschijnlijkheidsverdeling voor de boven- en benedenstroomse grenswaarden blijft een gevoelig punt waarvoor verder onderzoek vereist is. De beschouwing van niet-stationaire grenswaarden in deze studie omvat een lichte verbetering ten opzichte van het gebruik van een semi-stationaire opzet maar leidt nog steeds tot een overschatting van de overstromingskansen.
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