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Reliability-Based Design of Flood Defenses and Flood Defense Systems

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Abstract: *The combination of a quantitative reliability analysis with minimization of the construction cost is presented as a tool for the design of flood defenses. It is shown that the method can be used on the level of individual structures and on the level of systems of flood defenses. On the level of structures, the method provides the geometry of the structure as a function of the required safety level, the environmental conditions, and the cost of construction materials. On the level of systems, the method provides the optimal allocation of failure probabilities to individual structures to reach a required maximum flooding probability of the protected area.*

Keywords*: reliability analysis, probabilistic design, reliability-based design, optimization, flood defenses*

Introduction

The Netherlands is a low-lying country that depends on an extensive system of flood defenses for its existence. Following a major flooding disaster in 1953, statistical analysis of observed water levels was adopted as a tool to define the design requirements of flood defenses. Thus, the first elements of a reliability-based design approach entered the Dutch regulation as early as 1956 (Delta Committee, 1961). Over the years, considerable developments have taken place in the field of reliability-based design. The principles of reliability-based design were formulated by Turkstra (1962). Important principles for the calculation of reliability are due to Cornell (1969) and Hasofer and Lind (1974).

The current Dutch regulation with regard to the protection against flooding is still based on the statistical analysis of observed water levels as proposed by the Delta Committee. Over the years, it has become clear that an approach based on the uncertainty of water levels only omits important uncertainties relevant for the design of a flood defense structure, e.g. the uncertainty on the wave conditions in front of coastal flood defenses.

Over the past two decades, the Dutch Technical Advisory Committee for Flood Defenses (TAW) has recognized the discrepancy between the facts of nature and the methods for design of flood defenses. This has resulted in a research program aimed at the development of reliability-based design methods and ultimately risk-based design methods for flood defenses.

This paper describes the results of recent research into reliability-based design methods for large-scale flood defense systems.

Outline of Reliability Analysis of Flood Defenses

The area now called The Netherlands has seen permanent human occupation for the last 1,200 years. Due to sea level rise and land level recession, the people have been confronted with flooding with an increasing frequency. Efforts to protect settlements from flooding started locally. Locally constructed flood defenses were gradually extended and connected to the flood defenses of neighboring settlements. Thus, a number of areas protected by a closed ring of flood defenses came into existence. Such a ring or system of flood defenses is called a dike ring.

The first step in the reliability analysis of a dike ring is the definition of the main function and derived from that the definition of failure. Several functions may be allocated to a flood defense system, but the primary function is the retention of high water levels to prevent flooding of the area protected by the system. Therefore, the definition of failure is given by: water enters the protected area.

Failure of the dike ring occurs due to failure of one or more of the parts of the ring. Every part may fail due to a variety of failure modes. The second step in the reliability analysis of a dike ring therefore consists of a thorough qualitative risk analysis. Such an analysis consists of a breakdown of the ring into distinct elements (or structures) and an assessment of the individual failure modes

of the elements. The results can be conveniently represented in a fault tree. In the fault tree, the definition of failure is given in the top of the tree and the logical connection of this so-called top event to individual causes of failure is shown. The qualitative analysis should in principle be exhaustive. The reason for this is that any cause of failure that is not recognized in the qualitative analysis will also not be considered in the quantitative analysis that follows.

The third step in a reliability analysis of a dike ring is the quantification of the probability of failure on the level of individual failure modes. To this end, every individual failure mode has to be cast in a reliability function. The general form of a reliability function is given by

$$g(\mathbf{z},\mathbf{x}) = R(\mathbf{z},\mathbf{x}) - S(\mathbf{z},\mathbf{x}) \tag{1}$$

where R is the resistance of the structure; S is loading on the structure; $\mathbf{x}$ is the vector of stochastic input variables; $\mathbf{z}$ is the vector of design variables.

The reliability function is defined in such a way that negative values indicate failure of the structure by the failure described in the reliability function. The probability of occurrence of a failure mode written in the form of Equation 1 is given by

$$P_f(\mathbf{z},\mathbf{x}) = P\left(g(\mathbf{z},\mathbf{x}) < 0\right) = \int_{g(\mathbf{z},\mathbf{x})<0} f_{\mathbf{x}}(\mathbf{x})\,d\mathbf{x} \tag{2}$$

Equation 2 is the volume integral of the joint probability distribution of the stochastic input conditions $\mathbf{x}$ over the domain where the reliability function indicates that the failure mode occurs. An analytical solution of Equation 2 is generally not available. Over the years, a number of calculation methods have been developed. Once the probability of failure on the level of failure modes is quantified, several methods are available to calculate the probability of occurrence of the top event. See Melchers (1999) for an overview. More details on the application to coastal structures are given in Oumeraci et al. (2001) and Voortman (2002).

Method of Reliability-Based Optimization of Flood Defense Structures

Using the methods briefly outlined in the previous section, the probability of failure of a system of any complexity can be established as a function of the vector of design variables $\mathbf{z}$. The calculated failure probability can be used to judge the acceptability of a certain design alternative. If a maximum value of the failure probability is defined, the set of acceptable values of the design variables is given by:

$$D = \left\{\mathbf{z} \mid P_f(\mathbf{z}) \le P_{f;max}\right\} \tag{3}$$

where $P_{f;max}$ is the maximum acceptable value of the failure probability.

In general, Equation 3 describes an infinite number of acceptable geometries. Therefore, an additional requirement is necessary to limit the choice to one alternative. From the viewpoint of rationality, it appears appropriate to require the costs of protection to be minimal, provided the structure meets the pre-defined failure probability requirement. In other words, the appropriate geometry of the flood defense is found by solving the following minimization problem

$$\begin{aligned} &\min_{\mathbf{z}} I(\mathbf{z}) \\ &\text{s.t. } P_f(\mathbf{z},\mathbf{x}) \le P_{f;max} \end{aligned} \tag{4}$$

where I denotes the cost of the structure (see Table 2 for details).

In general the requirements of cost minimization and meeting the failure probability requirement are conflicting. This means that an increase of a design variable (for instance the crest level of a dike) decreases the failure probability but at the same time increases the cost of the structure. Therefore the minimization problem of Equation 4 generally leads to a unique solution, provided the space of the design variables is continuous. Figure 1 shows a simple example of reliability-based optimization of a flood defense structure. The thick line indicates the set of geometries that fulfill the failure probability requirement. The thin lines indicate the set of geometries with constant cost.

A software tool to solve the optimization problem of Equation 4 can be developed on the basis of readily available routines for reliability analysis and numerical optimization (Voortman, 2002).

Method of Reliability-Based Optimization of Flood Defense Systems

As stated before, the reliability of a Dutch dike ring can be calculated by splitting the ring in a number of individual sections or structures. Optimization of such a system can in principle be performed directly on the basis of Equation 4. In

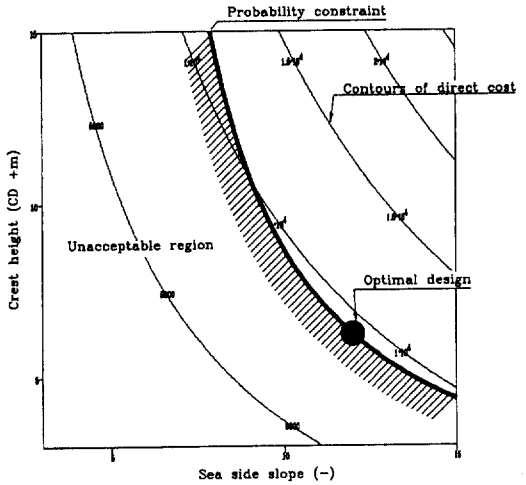

Figure 1. Graphical representation of reliability-based optimization

that case, the optimization problem is given by:

$$\min_{\mathbf{z}} I_{sys}(\mathbf{z}) = \sum_{n=1}^{N} I_n(\mathbf{z}) \quad \text{s.t. } P_{f;sys}(\mathbf{z},\mathbf{x}) \le P_{f;max} \tag{5}$$

where I_{sys} denotes the cost of protection on system level and $P_{f;sys}$ the probability of failure of the system.

The number of elements of the vector $\mathbf{z}$ is approximately N times larger than the number of elements of $\mathbf{z}$ in Equation 4. Solving the optimization problem on system level in this way directly leads to the optimal geometry of every structure in the ring. From the optimal geometry, the failure probability of every structure can be derived using quantitative fault tree analysis as outlined previously. This approach is referred to as the top-down approach, because the optimization problem is formulated on the level of the dike ring and the properties of the individual structures are derived from the full ring optimization.

The top-down approach is the most obvious method when thinking about optimization of large-scale systems. The approach is attractive if the number of design variables in the system is fairly limited. An example of the top-down approach is the optimization of a land reclamation project in the North Sea by Stroeve and Sies (2001). For most large-scale systems, the number of design variables is large and the top-down method then becomes less transparent and possibly hard to handle from a computational point of view. An alternative approach is found when noting the following properties of quantitative fault tree analysis:

- The probability of failure of an individual structure is a function of its own design variables, but not of the design variables of other structures;
- The probability of failure of the ring is a function of failure probabilities of the individual structures and of the dependence between them.
- Therefore, an alternative formulation of the reliability-based optimization of a dike ring is given by

$$\min_{\mathbf{p}} I_{sys}(\mathbf{p}) = \sum_{n=1}^{N} I_n(p_n) \quad \text{s.t. } P_{f;sys}(\mathbf{p},\mathbf{A}) \le P_{f;max} \tag{6}$$

where $\mathbf{p}$ is a vector of failure probabilities by structure.

At first sight, it seems that this formulation shifts the problem to establishing the cost of protection by individual structure as a function of its failure probability. This however can be performed by performing reliability-based optimization of every individual structure for a range of failure probability requirements. As observed before, the number of design variables in the optimization of an individual structure is considerably less than the total number of design variables. It is shown in Voortman (2002) that the bottom-up approach is computationally more efficient than the top-down approach in most cases. Furthermore, the approach appears to provide more insight in the properties of optimally designed flood defenses because it allows the analyst to deal with small partial problems first before embarking on a full system analysis. Therefore we prefer the bottom up method.

Description of the Case Study

The method of reliability-based design is applied to the 86 km long coastal flood defense system of the Dutch province Groningen (Figure 2).

The flood defense system of the province Groningen borders a shallow estuary called the Wadden Sea. The boundary between the Wadden Sea and the much deeper North Sea is formed by a chain of islands. In the figure, the boundary between Wadden Sea and North Sea is indicated by a dashed line. A reliability analysis of the flood defense system of the province Groningen is performed in TNO (1998). The system consists largely of dikes that can be broken down into a system of dike sections. The sections are defined in such a way that the hydraulic conditions along each section can be considered homogeneous. Therefore, a new section is defined every time there is a considerable change in the bottom level in front of the structure or if there is a change in orientation of the structure.

Inspection of Equation 2 shows that for a quantitative reliability analysis of flood defenses, the description of the joint probability distribution of environmental conditions is essential. Experience from earlier research, such as Van Gelder (2000) indicates that the reliability of coastal structures is generally dominated by the uncertainties on the hydraulic conditions. The hydraulic conditions relevant for the design of the flood defenses are water level, wave height, and wave period.

All three aspects of the hydraulic conditions are strongly influenced by the wind field over North Sea and Wadden Sea. Therefore a strong dependence between the three quantities is present that has to be accounted for in the description of the joint probability distribution (JPDF). Vrijling and Bruinsma (1980) developed the conceptual model for hydraulic conditions shown in Figure 3.

Following Vrijling and Bruinsma (1980), Voortman (2002) developed a model for the boundary conditions along the dike ring Groningen. Reliability-based design of the 11

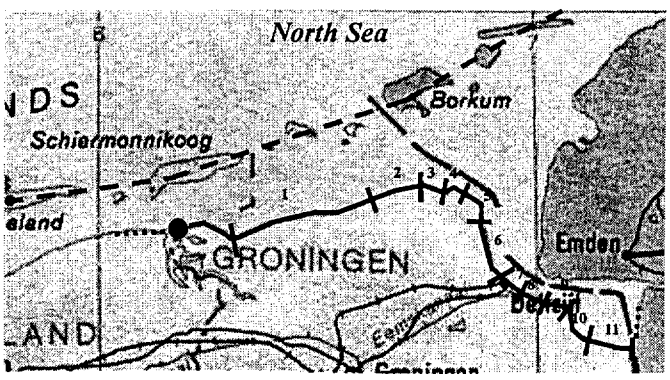

Figure 2. Case study area with flood protection system and breakdown in eleven sections

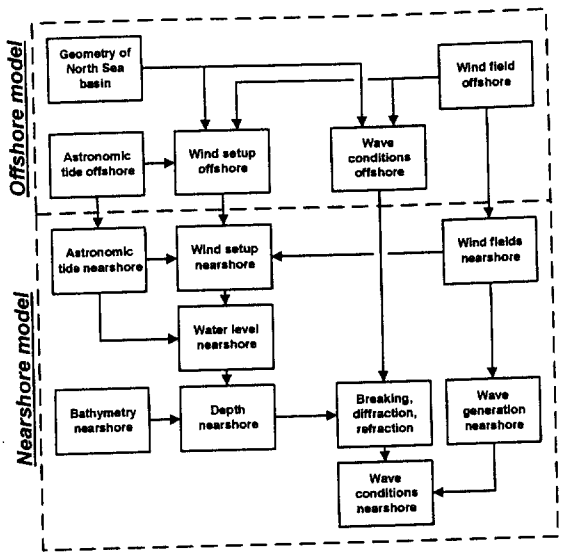

Figure 3. Conceptual model for hydraulic design conditions (after Vrijling and Bruinsma, 1980)

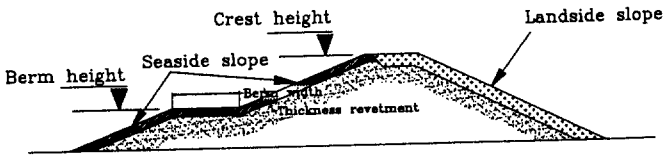

Figure 4. Schematized dike cross section with design variables

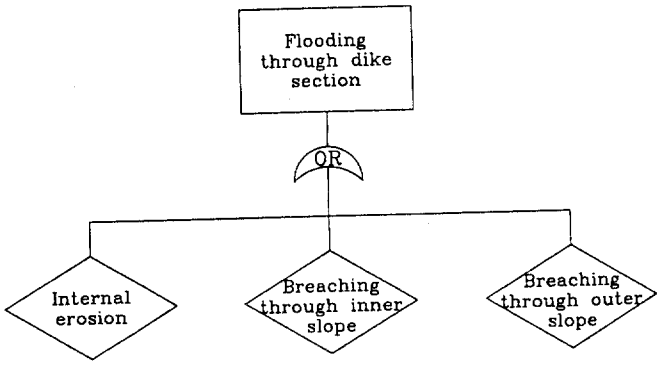

Figure 5. Fault tree of a dike section

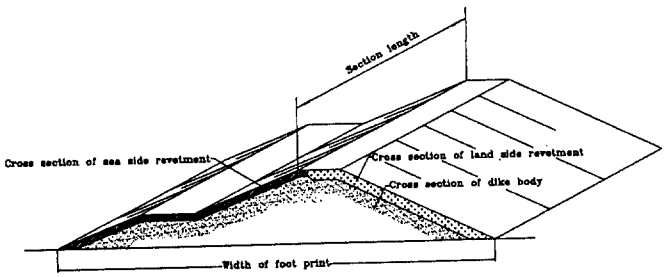

Figure 6. Cost components of a dike section

dike sections is performed using a set of six design variables to describe the geometry of the structure (Figure 4).

In the case study, the fault tree shown in Figure 5 is used to describe flooding through a dike section.

A large number of failure modes can be responsible for the three causes of failure shown in Figure 5. In this study, four failure modes are selected that are thought to a conservative description of failure of the structure (Table 1).

All failure modes used are "initiating" failure modes, which means that their occurrence is a prerequisite for the occurrence of failure of the structure. Between initial failure and full failure of the structure, some residual strength is present. However, the quantification of this residual strength is still a subject of research. It should be noted that for computational efficiency the simple model of Bligh is preferred over the Sellmeijer model.

The costs of the structure are quantified by considering the cost of distinct elements of the structure (Figure 6). The unit costs by cost component are given in Table 2.

Reliability-Based Design of Individual Flood Defenses

The sections of the flood defense system shown in Figure 2 differ in the elevation of the sea bottom in front of the structure, the position with respect to the offshore islands and the orientation of the structure. The optimization of the dike cross section will be shown for Locations 1 and 5 in Figure 2. Table 3 illustrates the differences in bottom level and hydraulic climate for the two sections. The table shows the bottom level in front of the section and the expected values of the hydraulic conditions corresponding to a wind speed with a probability of exceedence of 10^{-4} per year.

The different locations show a small difference in water level. More important is that at Eemshaven the wave condi-

Table 1. Overview of failure modes applied to quantify the probability of initial failure

Fault tree main branch	*Failure modes*
Internal erosion	Piping model of Bligh (TAW, 1999)
Breaching through inner slope	Wave overtopping (van der Meer and Janssen, 1995)
	Overflowing (included in definition of wave overtopping)
	Uplifting inner revetment (Voortman, 2002)
Breaching through outer slope	Failure of pitched block revetment (Hussaarts et al., 1999)

Table 2. Overview of cost variables

Cost component	*Price per unit*	*Remark*
Dike volume	4.6 €/m³	Price of filling sand including construction
Revetment outside	227.3 €/m³	Pitched concrete blocks. Price includes construction
Revetment inside	45.5 €/m³	Clay cover including construction
Space use	100 €/m²	Assumed value

Table 3. Hydraulic conditions conditional on a wind speed with probability of exceedence of 10^{-4} per year

	Section 1 Ommelander	*Section 5 Eemshaven East*
Bottom level (CD +m)	1.70	-1.56
Wind speed (m/s)	37.8	37.8
Water level (m)	6.46	6.55
Significant wave height (m)	2.30	3.79
Spectral peak period (s)	5.94	7.18

tions are more severe than at Ommelander. The lower bottom level at Eemshaven can be considered the main cause of the difference. For a given wind speed, there is still uncertainty on the hydraulic conditions which is fully accounted for in the reliability-based design procedure. The more severe hydraulic conditions are reflected in the dike profile that results from reliability-based optimization. Table 4 shows the design variables for optimized dike profiles at Ommelander and Eemshaven for a failure probability of 10^{-4} per year.

At both locations, the optimization indicates that it is not cost-effective to apply a sea side berm (Table 4). Increasing the width of the sea side berm reduces wave overtopping but has no effect on the stability of the revetment, since the steepest part of the profile is decisive in that respect. Flattening the sea side slope has a favorable effect both on wave overtopping and revetment failure. Due to the flatter slope, the necessary revetment thickness decreases, which reduces the cost considerably.

The effect of the more severe hydraulic conditions at Eemshaven is clearly reflected in the geometry of the optimal profile. All design variables (except for the berm geometry) that have an effect on the resistance of the structure against wave attack have a higher value at Eemshaven. The differences in the geometry of the dike section are also reflected in the construction costs by section. For the same probability of failure, the structure at Eemshaven is approximately 25 percent more expensive than the structure at Ommelander (Figure 7). Here only Case 1 is presented. For other cases studying e.g. the influence of the cost of space use, see Voortman (2002).

Reliability-Based Design of the Flood Defense System

Following the bottom-up approach shown in Equation 6, optimization of the full flood defense system can be performed

Table 4. Optimized geometries of two dike sections in the Dutch dike ring Groningen

	Section 1 Ommelander	*Section 5 Eemshaven East*
Crest level (CD +m)	11.8	13.0
Seaside slope (1:n)	9.4	9.6
Thickness revetment (m)	0.41	0.62
Sea side berm width (m)	0.0	0.0
Sea side berm level (m)	n.a.	n.a.
Land side slope (1:n)	3.0	3.0

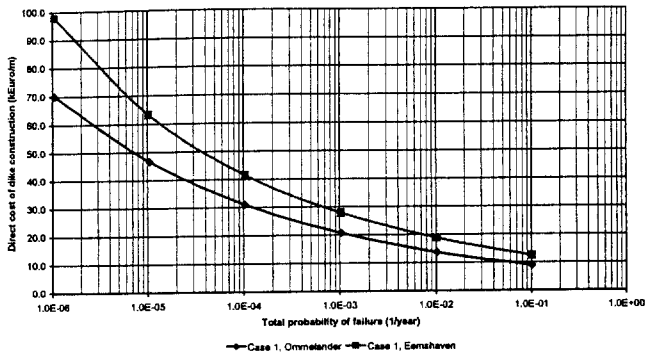

Figure 7. Cost of construction of two dike sections in Groningen as a function of failure probability

in the space of the failure probability by dike section. To this end, the cost of protection by dike section should be established as a function of the failure probability by section. This can be achieved by first optimizing each of all eleven dike sections of the dike ring Groningen for a range of failure probabilities. For every combination of location and probability of failure, the influence factors are obtained from the level II reliability calculation (Thoft-Christensen and Baker, 1982) that is an integral part of the optimization procedure. The influence factors can be used to quantify the dependence between the structures by a correlation coefficient (Voortman, 2002). Thus, sufficient information is available to establish the probability of failure of the system of dike sections by means of the method of Hohenbichler and Rackwitz (1983). The cost on system level is found by adding the costs of the individual structures. Optimization of the failure probability by section can now be performed. From a practical point of view, it can be convenient to have one design failure probability for all individual sections. The cost of protection on system level is shown in Figure 8 for both options. In addition, the upper and lower bounds of the cost on system level are shown. The upper and lower bound are obtained by analyzing the costs under the assumption that the system probability of failure equals one of the fundamental bounds (maximum or summation of the element failure probabilities) proposed by Cornell (1967).

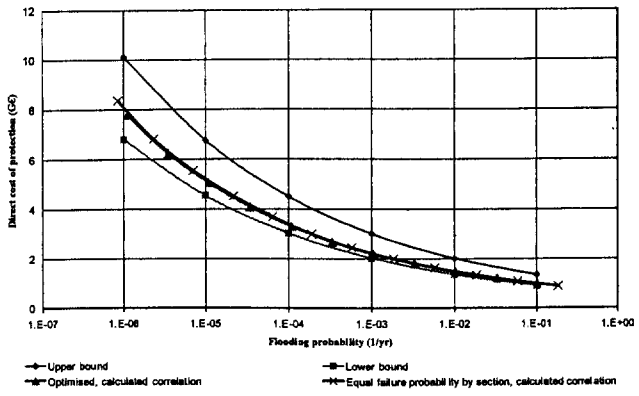

Figure 8. Cost of construction of an 86 km long dike system in the Dutch province Groningen as a function of flooding probability

From the result it can be concluded that for an accurate estimate of the cost on system level, it is necessary to consider the correlation between the sections. This is illustrated by the large difference between the upper and lower bound estimates of the cost of protection. Adopting the same design probability of failure by section for all sections increases the costs on system level by 1.5 percent in comparison to the mathematical cost minimum. In the current case, this small cost increase appears to be outweighed by the practical advantages of having the same design failure probability for every structure. Finally, it should be stressed that cost of construction is minimized given the failure probability. An optimization of the construction cost versus the risk of flooding is ongoing research.

Discussion and Conclusions

The method of reliability-based design of flood defenses enables design optimization for a pre-defined failure probability of the structure or system of structures. The method is able to deal with all relevant failure modes and all relevant uncertainties. Interesting insight is provided in the relative contributions of the uncertainties inherent as well as epistemic. Both are handled equally well. On the level of individual flood defenses, the method provides insight in the optimal geometry of the structure. On the level of systems of flood defenses, the method can be used to establish the optimal failure probability for individual structures, given a failure probability requirement on system level.

When applied to individual flood defenses, the results indicate that the optimal geometry is strongly influenced by interaction between failure modes. Interaction between failure modes takes place through design variables and boundary conditions that have influence on more than one failure mode at the same time. The results suggest that in design optimization, all relevant failure modes should be accounted for simultaneously.

Reliability-based design of a system of flood defenses illustrates that the dependence between elements of the system has an important influence on the system probability of failure and therefore on the cost estimate. When applied to the flood defense system of the Dutch province Groningen, it is shown that a practical choice of design failure probability by section can be adopted without causing a large deviation from the optimal system design.

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