RISK-BASED DESIGN OF LARGE-SCALE FLOOD DEFENCE SYSTEMS

H.G. Voortman[1], P.H.A.J.M. van Gelder[2] and J.K. Vrijling[3]

Abstract: On the basis of quantitative reliability analysis of flood defence systems, two design methods are developed. The first is reliability-based design, where the optimal geometry of a flood defence system is obtained by minimising the cost of construction under a constraint on the probability of flooding of the protected area. Reliability-based design is an integral part of the second design method, risk-based design. In risk-based design, the appropriate flooding probability of a protected area is obtained by comparing the cost of protection with the risk reduction that is obtained. A case study illustrates the application of both methods.

INTRODUCTION

The Netherlands is a low-lying country that depends on an extensive system of flood defences for its existence. Clear defined design requirements for the flood defence systems are necessary. Since 1956, the design requirements are given as prescribed exceedance frequencies of the water level in front of the structure (Delta Committee, 1961). At the time, this approach revolutionised the design and safety evaluation of flood defences in the Netherlands. With the statistical analysis of observed water levels, elements of probabilistic methods entered hydraulic engineering in the Netherlands for the first time.

Since the Delta Committee, probabilistic methods have been further developed. The fundamentals were developed by Turkstra (1962). Today, a number of tools for reliability-based design are now available to the hydraulic engineer (see Oumeraci et al, 2001 for an overview). As a result of the development of probabilistic methods, it is now recognised that:

1 Risk analysis specialist, ARCADIS, Postbus 220, 3800 AE, Amersfoort, The Netherlands, H.G.Voortman@arcadis.nl

2 Assistant professor of probabilistic design, Delft University of Technology, Faculty of Civil Engineering and Geosciences, Stevinweg 1, NL-2628 CN, Delft, The Netherlands, P.H.A.J.M.vanGelder@ct.tudelft.nl

3 Professor of hydraulic structures and probabilistic design, Delft University of Technology, Faculty of Civil Engineering and Geosciences, Stevinweg 1, NL-2628 CN, Delft, The Netherlands, J.K.Vrijling@ct.tudelft.nl

- Not only the uncertainty on the water level influences the reliability of a flood defence, but also uncertainties on wave conditions and properties of the structure itself;
- A variety of failure modes may cause failure of a flood defence structure.

Conceptually, all tools are available to deal with a multitude of uncertainties and a multitude of failure modes (van Gelder, 1999). There is however a need to develop a clear methodology to put the concept to work for the design of large-scale flood defence systems.

The Delta Committee recognised that the appropriate amount of flood protection of an area depends on the value of the protected area and on the cost of protection (van Dantzig, 1956). This was one of the fundamentals on which the Delta Committee based its recommendation for the safety levels that still prevail in current Dutch regulation. However, since the 1950's the Dutch economy and population have grown considerably. It is therefore thinkable that an update of the safety levels is necessary. In the light of the now available knowledge of probabilistic methods, an update of the exceedance frequency of design water levels only will not be acceptable. It is for this reason that over the last two decades, the Dutch Technical Advisory Committee for Water Defences (TAW) has worked towards a risk-based design method for flood defences. This paper introduces the results of recent research into the methodology of risk-based design of large-scale flood defence systems. An extensive description is available in Voortman (2002).

OVERVIEW OF RELIABILITY ANALYSIS OF FLOOD DEFENCE SYSTEMS

Definition of dike rings

The area now called the Netherlands has seen permanent human occupation for the last 1200 years. Permanent settlements emerged and soft soils were artificially drained to enable farming. Drainage of soft soils led and leads to land level recession. Furthermore, the sea level was (and is) rising gradually. The combination of human intervention and a changing environment made the Netherlands more and more vulnerable to flooding. Indeed, the first major floodings took place in the early 1100s (v.d. Ven, 1996). These events initiated the construction of flood protection. First in the form of dwelling mounds, later in the form of water retaining structures (dikes). In the beginning the construction of flood protection was a local initiative. However, people soon discovered that local flood protection is generally not effective, simply because water can still enters the area from neighbouring (unprotected) areas. Therefore, local flood defences were extended until they formed closed dike rings. The dike ring is still the basis of the current Dutch flood defence policy. The safety levels mentioned in the previous section are defined for 53 clearly defined dike rings.

Reliability analysis of dike rings

A dike ring has the character of a chain of individual flood defence structures that may differ in:

- Type of structure (dike, dune, ship lock, discharge sluice);
- Loading on the structure (river regime, sea regime, orientation with respect to wind direction);
- Geometry of the structure.

The goal of a reliability analysis is to establish the probability of failure of the dike ring. The analysis of such a large-scale system can be split in the following steps:

1. Definition of the primary function of the system and definition of failure;

2. Break-down of the event "failure of dike ring" into failures of individual structures and establishing the logical connections between them;
3. Break-down of the events "failure of structure" into failure modes and establishing their logical connections;
4. Quantifying the probability of occurrence of every failure mode;
5. Combining the probabilities of occurrence of failure modes to the failure probability per structure;
6. Combining the probabilities of failure by structure to the failure probability of the ring.

Steps 1 through 3 constitute the qualitative reliability analysis of the dike ring. In principle the qualitative analysis should be aimed at identifying all causes of failure. The reason for this is that any cause of failure that is missed in the qualitative analysis will also not be analysed in the quantitative analysis that follows. The result of the qualitative analysis is conveniently presented in a fault tree (figure 1).

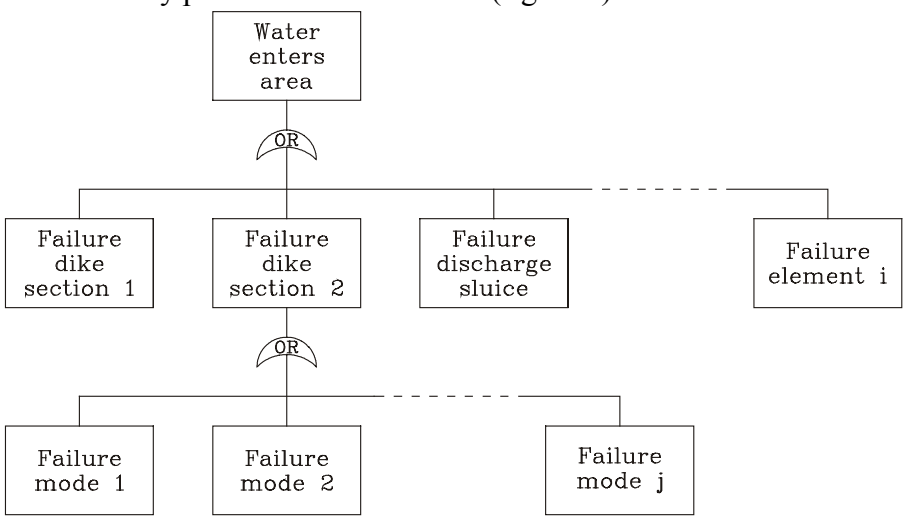

Figure 1: Example fault tree of a dike ring

Quantification of the probability of flooding starts with the definition of reliability functions for the failure modes in the lowest level of the fault tree. A form of a reliability function that covers a large number of cases 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 the loading on the structure, $\mathbf{z}$ is a vector of design variables describing among others the geometry of the structure and $\mathbf{x}$ is a vector of random variables like wind speed, water level and wave height.

The occurrence of the failure mode described by equation (1) is indicated by negative values of the reliability function. The probability of occurrence of every failure mode is given by:

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

Where $f_{\underline{\mathbf{x}}}(\mathbf{x})$ denotes the joint probability distribution of random input conditions.

The integral in equation (2) can generally not be solved analytically. A variety of methods is available to quantify the probability of failure (see for instance Melchers, 1999 or Oumeraci et al, 2001). Once the probability of occurrence of every failure mode is established, the probability of failure of every structure and of the system can be found. Upper and lower bound estimates of this so-called system probability of failure are available (see Ditlevsen, 1979 for an overview). Furthermore, there are a number of methods that provide approximations to the probability of failure of the system. The choice of the method is sometimes limited by practical requirements (Voortman et al., 1998; Voortman, 2002). In this study, the method proposed by Hohenbichler and Rackwitz (1983) is applied.

DESIGN METHODS FOR FLOOD DEFENCE SYSTEMS

Reliability-based design

If the geometry of every structure is known and the joint probability distribution of load and strength variables is quantified, the probability of failure of the system of flood defences can be found. However, very often the question is reversed. For a pre-defined failure probability a geometry of the structure needs to be found. For a fixed value of the probability of failure, the set of acceptable design alternatives is conceptually given by:

$$D = \left\{ \mathbf{z} \mid P_{flood}\left(\mathbf{z}\right) \leq P_{flood;max} \right\} \tag{3}$$

Where $P_{flood;max}$ denotes the maximum acceptable probability of flooding.

Equation (3) in principle provides an infinite set of acceptable design alternatives. To find a unique solution, other requirements have to be accounted for. It appears rational to look for minimisation of the construction costs of the system, provided the system fulfils the pre-defined requirement on the failure probability. This leads to the concept of reliability-based optimisation or reliability-based design. For a single structure with two design variables, the process is visualised in figure 2.

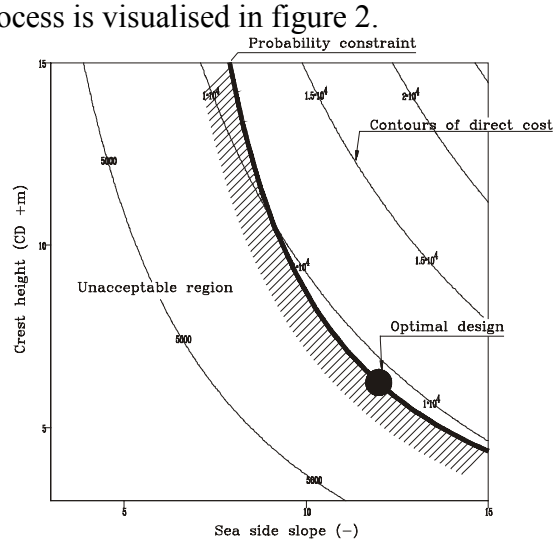

Figure 2: Overview of reliability-based optimisation

Mathematically, reliability-based design is performed by solving the following minimisation 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 direct cost of the structure.

In most cases, the partial derivatives of the cost and failure probability to the design variables have constant and opposite signs. In that case a unique solution to equation (4) can be found by application of numerical minimisation routines. In a design process, different structural concepts may be applicable. In that case the optimisation procedure should be carried out for every concept after which a comparison of the costs for the same failure probability can be used to make a final choice (see Laenen, 2000 for an example).

Optimisation in the form of equation (4) may in principle be performed on the level of the full system and on the level of individual structures. Applied to systems, the optimisation problem is formulated on system level and the failure probabilities and geometries of individual structures follow from the system optimisation. Therefore, this approach is denoted the top-down approach. In most cases the top-down approach leads to an optimisation problem with a large number of dimensions that is sometimes computationally hard to handle. The approach also has a tendency to become a "black box", meaning that as an analyst it is very hard to understand the result of the optimisation if the number of dimensions in the design problem is large.

An alternative approach is based on the notion that the probability of failure on system level is a function of the failure probabilities of the individual structures and of the dependence between them. Therefore, optimisation of the system can be written as:

$$\begin{aligned} &\min_{\mathbf{p}} I_{sys}(\mathbf{p}) = \sum_{n=1}^{N} I_n(p_n) \\ &\text{s.t. } P_{f;sys}(\mathbf{p},\mathrm{A}) \le P_{f;max} \end{aligned} \tag{5}$$

Where $\mathbf{p}$ denotes a vector of failure probabilities and A a matrix of influence factors and I_n the direct cost of structure n.

At first sight it appears that this approach only shifts the problem to the definition of the cost by structure as a function of its failure probability. However, this function can be achieved for every structure by performing reliability-based optimisation for a range of required failure probabilities. Since in this approach the structures are optimised first and the system optimisation is based on the results by structure, the method is denoted the bottom-up approach. The bottom-up approach is generally more transparent than the top-down approach. An additional advantage is that the method is computationally more efficient in most cases (Voortman, 2002).

Risk-based design

Reliability-based design depends on the availability of a pre-defined failure probability requirement. In a situation where acceptable safety levels are defined in regulation, the methods outlined in the previous section are sufficient to obtain a cost-effective design that fulfils the requirements. In some cases, the required failure probability is not defined. This is the case when:

- A design is made for a situation where no regulation is available. In the Netherlands this situation occurs for example if a structure is located outside a dike ring;
- When an analysis is performed in a process of (re)defining required safety levels.

The acceptable probability of failure can be defined by comparing the cost of protection to a characteristic value of the consequences of flooding. In a purely economic sense this leads to risk-based cost-benefit analysis. Applications in coastal engineering have been published by Bakker and Vrijling (1980), Burcharth et al (1995), Voortman et al (1998, 1999a, 1999b), Vrijling et al (1998) and Stroeve and Sies (2001). A few alternatives to the cost-benefit approach are shown in Voortman et al. (2001) and Voortman (2002).

In risk-based design, a model is established that provides a measure of the effectiveness of the protection system as a function of its failure probability. The two main components of such a model are the cost of protection and a characteristic value of the consequences of flooding, both as a function of the probability of failure. An example is the cost-benefit model for flood protection that is given by:

$$B_{ref}\left(P_{flood},T\right)=-I_{sys}\left(P_{flood}\right)+\sum_{t=0}^{T}\frac{b\left(1+r_e+i\right)^t}{\left(1+r\right)^t}-\sum_{t=0}^{T}\frac{P_{flood}\left(b+d\right)\left(1+r_e+i\right)^t}{\left(1+r\right)^t} \qquad (6)$$

Where T is the reference period, I_{sys} the cost of protection, b the yearly economic benefits in undisturbed conditions, d the direct damage in case of flood, r_e rate of economic growth, i inflation and r interest rate.

In the right-hand side of equation (6) three terms can be distinguished. The first term denotes the cost of protection as a function of the flooding probability. The second term denotes the economic benefits, capitalised over the reference period T. The third term denotes the expected value of the economic loss in case of flooding, also capitalised over the reference period. The economic loss consists of direct loss and indirect loss. Direct loss is a consequence of contact with the flood water. Indirect loss or production loss is a consequence of the direct loss. The difference may be clarified by an example. If a factory is flooded, it may suffer a loss because flooded machinery may need to be replaced. This is denoted the direct loss. The loss of the machinery and the fact that the factory can not work for some time reduces the benefits of the factory. The loss of benefits is denoted indirect loss.

The cost of protection as a function of the flooding probability can be established by application of the reliability-based design method on system level. Estimates of the invested capital in the area and the yearly production may be found in databases that are administered by statistical agencies (see for instance the Dutch Bureau of Statics on internet).

CASE STUDY: THE DIKE RING GRONINGEN

Overview of the case study and input

The application of reliability-based design and risk-based design is demonstrated on a case study. The goal of the case study is to establish the economic optimal design of the 86 km long coastal flood defence system of the Dutch province Groningen, located in the south of the North Sea. The coast line of the dike ring 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 offshore islands. Figure 4 shows the case study area in detail. A dashed line indicates the approximate position of the boundary between North Sea and Wadden Sea.

Figure 3: Overview of case study area

Following the steps of the qualitative reliability analysis, the flood defence system is broken down in 11 distinct dike sections. The break-down corresponds to the break-down developed in TNO (1998).

Past experience in a number of projects indicates that the reliability of coastal flood defences is dominated by the uncertainties on the hydraulic conditions in front of the structure. In Voortman (2002) a method is proposed to describe the joint probability distribution function (JPDF) of hydraulic conditions on the basis of wind statistics and parametric physical models (see also Webbers et al, 2002). The physical models provide the water level, significant wave height and spectral period on a number of locations as a function of the wind speed over the North Sea. The model is calibrated to field data where possible.

Reliability-based design of the 11 dike sections is performed using a set of six design variables for each section (figure 5).

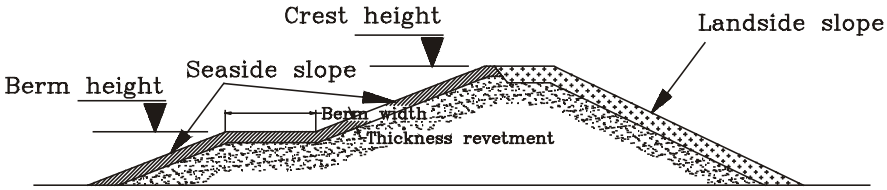

Figure 4: Schematised dike cross section and design variables

The fault tree used to describe failure of a dike section is given in figure 6.

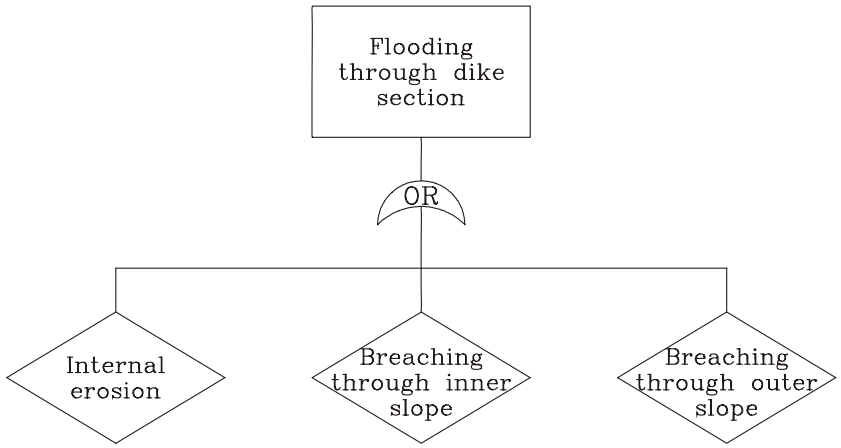

Figure 5: Fault tree for a dike section

The three causes of failure in figure 6 can be considered summaries of a large number of underlying failure modes. In this study, conservative models have been chosen for every branch of the fault tree. An overview is given in table 1.

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 (v.d. 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)

All failure modes describe the beginning of failure of the structure. An analysis of the probability of failure with this set of failure modes leads to an estimate of the probability of failure initiation, which is an upper bound of the actual probability of failure (Voortman, 2002).

The costs of construction of a dike section are obtained by calculating the cost of three distinct elements in the dike cross section (figure 7).

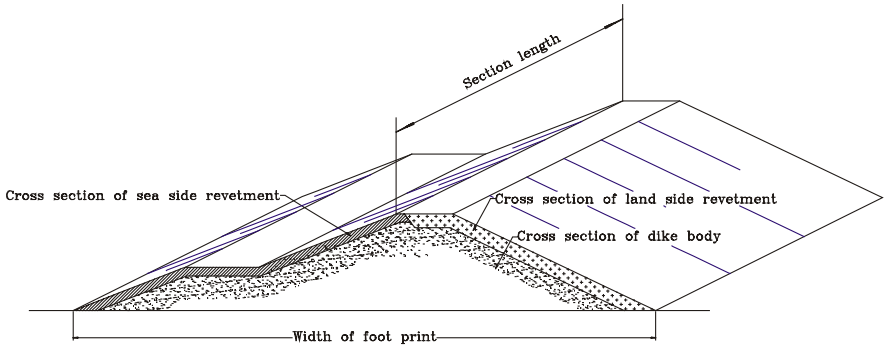

Figure 6:Cost components of a dike section

The unit costs by cost component are obtained from the engineering division of the Dutch Ministry of Public Works and Water Management and are given in table 2.

Table 2: Overview of cost variables

Cost component	Price per unit	Remark
Dike volume	4.6 €/m^3	Price of filling sand including construction
Revetment outside	227.3 €/m^3	Pitched concrete blocks. Price includes construction
Revetment inside	45.5 €/m^3	Clay cover including construction
Space use	100 €/m^2	Assumed value

For risk-based design, estimates of direct and indirect damage in case of flooding are necessary. Furthermore, a number of macro-economic parameters need to be quantified. Table 3 provides an overview.

Table 3: Input for the risk calculation in the dike ring Groningen

Parameter	Description	Value	Remark
d_0	Maximum direct damage	G€ 34,-	Taken from PICASO study*
b_0	Maximum indirect damage	G€ 14.40	Value of 1998#
r	Interest rate	0.07	Average over 1960-2001#
r_e	Rate of economic growth	0.03	Average over 1960-2001#
i	Inflation	0.02	Average over 1960-2001#
T	Reference period	100	Assumed value

*: RWS, 2001
#: Data obtained from the database of the Dutch Central Bureau of Statistics.

Analysis of the cost of protection on system level

Application of the reliability-based design method to all dike sections leads to the cost of construction as a function of the failure probability by structure. Figure 8 provides an overview.

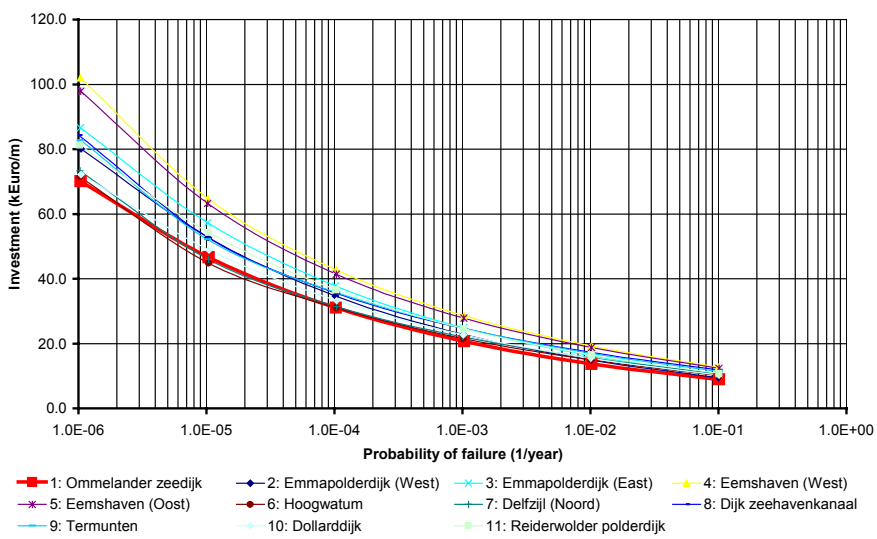

Figure 7: cost of protection by dike section

The hydraulic loads on a dike section are location-dependent. In general, heavier loaded structures show a higher level of construction costs for the same probability of failure. The most expensive dike sections are located near the Eems harbour where a deep tidal channel enters the estuary. The cheapest sections are sections 1 through 3 that face a large shallow area and are sheltered by offshore islands. Deeper in the Eems-Dollard estuary the maximum water levels during storm conditions increase, which leads to an increase of the location-dependent cost of protection.

For every combination of failure probability and dike section, the cost of protection and the influence factors are available. Thus, sufficient information is available to perform system optimisation by the bottom-up strategy outlined earlier in this paper. A number of cost estimates can be made on the basis of this information. The first two estimates are upper and lower bounds on the cost of protection on system level that are found if the flooding probability is considered equal to one of the fundamental bounds of Cornell (1967). The dependence between different dike sections can be quantified by a correlation coefficient that is a function of the influence factors by section (Voortman, 2002). The result of the optimisation using calculated dependence provides the third cost estimate that should obviously lie between the upper and lower bound estimates. Optimisation of the failure probability per section will lead to location-dependent variation of the target failure probability. From a practical point of view a single value for all structures would be favourable. Adopting a fixed design failure probability by section leads to the fourth estimate of the cost of protection on system level. Figure 8 shows the results.

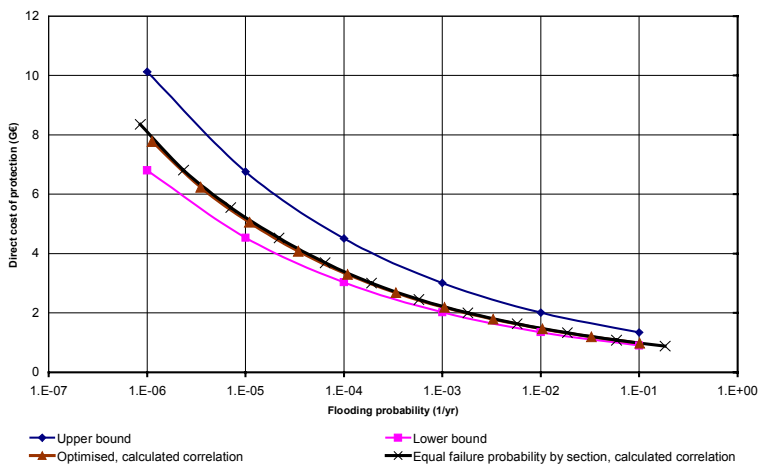

Figure 8:Four estimates of the cost of protection on system level

The upper bound estimate is approximately 40 % higher than the lower bound estimate. It is therefore necessary to perform an analysis accounting for the calculated dependence between the sections. As required, this estimate lies between the upper and lower bounds. As a function of decreasing flooding probability, the dependence between the dike sections decreases. This leads to an increasing deviation of the cost estimate from the lower bound. Adopting a single design failure probability by section leads to a cost increase of at maximum 1.5 % with respect to the mathematical optimum. Considering the major practical advantages of a single design failure probability, it appears to be the best solution in this case.

For use in risk-based design, it is convenient to establish an analytical expression for the cost of protection on system level. A suitable approximation is given by:

$$I_{sys}\left(P_{flood}\right) = 10^{-a\log\left(P_{flood}\right)+b} \tag{7}$$

Where P_{flood} is the flooding probability and I_{sys} the cost of protection in G€. Equation (7) is calibrated on the basis of results for flooding probabilities of 10^{-1} per year to 10^{-6} per year, in which case a equals 0.18 and b 8.81.

Risk-based design of the protection system

With the estimate of the cost of protection obtained in the previous section and the estimates of the damage in case of flooding, economic optimisation of the dike ring Groningen can be performed. Figure 9 shows the cost-benefit model (equation 6) for the input conditions of the dike ring Groningen.

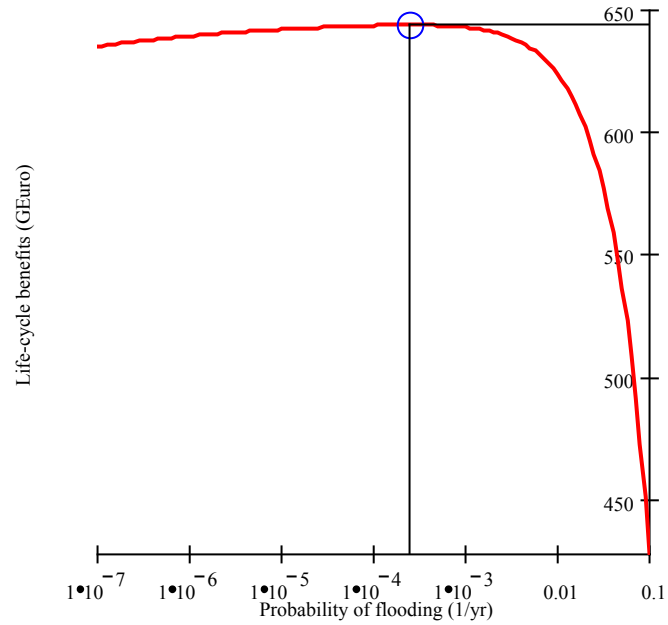

Figure 9:Cost-benefit analysis applied to the dike ring Groningen

The flooding probability that maximises the life-cycle sum of costs and benefits equals $2.4 \cdot 10^{-4}$ per year. Taking equal failure probability by section, this flooding probability can be obtained by adopting a design failure probability on section level of $1.3 \cdot 10^{-4}$ per year. It is important to note that the design failure probabilities derived in this study are incomparable to the exceedance frequency of the design water level, as is used in current regulation. The failure probabilities derived in this study summarise the effects of multiple failure modes, a multitude of uncertainties and location dependence of the boundary conditions. The optimal geometry of every individual dike sections is derived by applying reliability-based optimisation for a failure probability of $1.3 \cdot 10^{-4}$ per year.

CONCLUSIONS

The combination of quantitative reliability analysis with cost optimisation provides an effective tool for the design of large-scale flood defence systems. The method can be used to establish a cost-effective design for a pre-defined probability of flooding or as an integral part of a risk-based design method. All relevant aspects of the natural environment, economic aspects of the design, the consequences of flooding and the technical properties of design alternatives are incorporated in one consistent framework for decision-making.

Application to a case study shows that the method fulfils all expectations. Insights can be gained on the effect of location-dependent loading on the design and cost of a structure and on the appropriate level of protection of an area in view of the cost of protection and the consequences of flooding.

ACKNOWLEDGEMENTS

The research described in this paper was performed while the first author was employed as research assistant at Delft University of Technology. The project was part of the Delft Cluster program (project DC 02.02.03) and was supported by the Dutch Ministry of Public Works and Water Management (contracts RKZ-642 and DWW-1528).

REFERENCES

Bakker, W.T; Vrijling J.K (1980); *Probabilistic design of sea defences*, Proceedings International Conference on Coastal Engineering

Burcharth, H.F; Dalsgaard Sørensen, J; Christiani, E (1995), *Application of reliability analysis for optimal design of vertical wall breakwaters*, Proceedings of the International Conference on Coastal and Port Engineering in Developing Countries (COPEDEC)

Cornell, C.A (1967), *Bounds on the reliability of structural systems,* Journal of the Structural Division, ASCE, 93 - pg. 171-200

Dantzig, D. van (1956), *Economic Decision Problems for Flood prevention*, Econometrica nr. 24, pg 276-287

Delta Committee (1961a), *Final report and interim-advices (in Dutch)*, Delta Report, considerations concerning storm surges and tidal movements, volume I

Ditlevsen, O. (1979), *Narrow reliability bounds for structural systems*, Journal of Structural Mechanics, Pg. 453-472

Gelder, P.H.A.J.M. van (1999), *Statistical Methods for the Risk-based design of Civil Structures*, Phd-thesis Delft University of Technology

Hohenbichler, M; Rackwitz, R. (1983), *First-order concepts in system reliability*, Structural Safety, pg. 177-188

Laenen, K.C.J (2000), *A probabilistic model for the comparison of two breakwater types based on economic optimisation (in Dutch)*, M.Sc. thesis Delft University of Technology

Melchers, R.E (1999), *Structural reliability, analysis and prediction (2nd ed.)*, Wiley, Chichester

Oumeraci, H; Allsop, N.W.H; Groot, M.B. de; Crouch, R; Vrijling, J.K; Kortenhaus, A; Voortman, H.G (2001), *Probabilistic Design Tools for Vertical Breakwaters*, Balkema, Rotterdam

Rijkswaterstaat (2001), *PICASO, pilot case flooding risk (in Dutch)*

Stroeve, F.M. and Sies, E.M. (2001), *Integral optimisation of land reclamation in the North Sea*, Proceedings IABSE-conference safety, risk and reliability - trends in engineering

TNO (1998), *Case study GroFriland,* Report prepared for TAW

Turkstra (1962, 1970), *Theory of structural design decisions,* University of Waterloo

Ven, G.P. van de (ed.) (1996), *Man-made lowlands, history of water management and land reclamation in the Netherlands (3rd revised edition)*, Matrijs publishers

Voortman, H.G; Kuijper, H.K.T; Vrijling, J.K (1998), *Economic optimal design of vertical breakwaters*, Proceedings 26th International Conference on Coastal Engineering (ICCE)

Vrijling, J.K; Gopalan, S; Laboyrie, J.H; Plate, S.E. (1998a), *Probabilistic Optimisation of the Ennore Coal Port,* Proceedings of the International Conference on Coastlines, Structures and Breakwaters, pg 135-147, Thomas Telford, London

Voortman, H.G; Kuijper, H.K.T; Vrijling, J.K. (1999a), *Influence of Subsoil Properties on the Optimal Design of Caisson Breakwaters*, Proceedings of the International Conference on Coastal and Port Engineering in Developing Countries (COPEDEC)

Voortman, H.G; Vrijling, J.K; Boer, S; Kortlever, W.C.D. (1999b), *Optimal Breakwater Design for the Rotterdam Harbour Extension*, In: Goossens (ed.), Proceedings of the 9th annual conference of the Society for Risk Analysis Europe

Voortman, H.G; Pandey, M.D; Gelder, P.H.A.J.M. van; Vrijling, J.K (2001), *Definition of acceptable risk in flood-prone areas*, Corotis, R.B; Schuëller, G.I; Shinozuka, M. (ed.) (ed.) Proceedings of the International Conference on Structural Safety and Reliability (ICOSSAR)

Voortman, H.G. (2002), *Risk-based design of large-scale flood defence systems*, Phd-thesis Delft University of Technology

Webbers, P.B; Voortman, H.G; Gelder, P.H.A.J.M. van; Vrijling, J.K (2003), *Multi-variate description of hydraulic boundary conditions for the Rotterdam harbour extension*, Proceedings of the International Conference on Coastal Engineering 2002

INTERNET RESOURCES

Dutch Central Bureau of Statistics - www.cbs.nl

RISK-BASED DESIGN OF LARGE-SCALE FLOOD DEFENCE SYSTEMS

H.G. Voortman[4], P.H.A.J.M. van Gelder[5] and J.K. Vrijling[6]

Keywords:

Risk-based design
Flood defences
Safety
Reliability
Failure probability
Failure mechanisms
Fault tree
Cost benefit analysis
Economic optimisation

4 Risk analysis specialist, ARCADIS, Postbus 220, 3800 AE, Amersfoort, The Netherlands, H.G.Voortman@arcadis.nl

5 Assistant professor of probabilistic design, Delft University of Technology, Faculty of Civil Engineering and Geosciences, Stevinweg 1, NL-2628 CN, Delft, The Netherlands, P.H.A.J.M.vanGelder@ct.tudelft.nl

6 Professor of hydraulic structures and probabilistic design, Delft University of Technology, Faculty of Civil Engineering and Geosciences, Stevinweg 1, NL-2628 CN, Delft, The Netherlands, J.K.Vrijling@ct.tudelft.nl