Application of reliability-based flood defence design in the UK

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Increasing concern about recent and potential impacts of flooding in the UK are leading to the adoption of risk-based methods for planning, appraisal, design and operation of flood defences. Probabilistic methods for assessment and design of flood defences are relatively well developed in the Netherlands because of the potentially devastating impacts of flooding. This paper describes a test application of reliability-based design tools developed for dike rings in the Netherlands to the flood defence system at the Caldicot Levels in South Wales. Although constrained by data limitations (even at this relatively data-rich site, by UK standards) the reliability method provides estimates of the probability of failure of the flood defence system, identifies weak system components and identifies which parameters contribute most to the probability of failure. However, for reasons described in this paper, reliability methods developed in the Netherlands are not universally applicable in the UK context.

Key words: PC-Ring, failure modes, flood defence system, probability of failure, weakest links

1 Introduction

In the Netherlands as well as in the UK, flood protection policy is currently undergoing changes towards a risk-based approach of flood defences. A risk-based safety approach takes into account the strength and loading conditions of the flood defence system as part of the probability of inundation as well as the consequences of inundation in case of failure of the flood defence system. Vrijling (2001) points out that a risk-based analysis of flood defence systems can result in the identification of the system's weak areas and can therefore enable the decision-maker to target improvement schemes and maintenance activities. Another advantage is that in case of large scale flood defence improvements the decision-maker can compare different design options in terms of actual risk reduction and the costs which are associated with the improvement options. In the light of the shift to the risk-based safety approach the UK Environment Agency and the Department of the Environment, Food and Rural affairs, which together have the responsibility for flood defence policy and implementation in the UK have launched a research and development project called RASP: Risk Assessment of flood and coastal defences for Strategic Planning. This project aims to develop tiered methodologies for risk assessment of flood defence systems: a high level methodology supporting national policy making, an intermediate level supporting regional policy making and a detailed level approach supporting policy making at the scale of one flood defence system. Table 1 provides an overview of these tiered methodologies.

Table 1: Tiered risk assessment methodologies in RASP

Level	Decisions to inform	Data sources	Methodologies
High	- National assessment of economic risk, risk to life or environmental risk - Prioritisation of expenditure	- Defence types - Condition grades - Standard of service - Indicative flood plain maps - Socio-economic data - Land use mapping	- Generic probabilities of defence failure - Assumed dependency between defence sections - Empirical methods to determine likely flood extent
Inter-mediate	*Above plus* - Flood defence strategy planning - Regulation of development - Prioritisation of maintenance - Planning of flood warning	*Above plus* - Defence crest level and other dimensions where available - Joint probability load distributions - Flood plain topography - Detailed socio-economic data	*Above plus* - Probabilities of defence failure from reliability analysis - Systems reliability analysis using joint loading conditions - Modelling of limited number of inundation scenarios
Detailed	*Above plus* - Scheme appraisal and optimisation	*Above plus* - All parameters required describing defence strength - Synthetic time series of loading conditions	*Above plus* - Simulation based reliability analysis of system - Simulation modelling of inundation

This paper describes a test application of reliability-based design tools developed for dike rings in the Netherlands to the Caldicot Levels' flood defence system in South Wales. This first application of PC-Ring in the UK supports an evaluation of the appropriateness of this reliability method for flood defences as part of the detailed level methodology in RASP.

The main working-method is derived from CUR report 190 (1997) and consists of carrying out the reliability analysis by taking the following steps (see Figure 1 for detailed approach):

- Definition of the Caldicot Levels' flood defence system and its components
- Analysis of the failure modes connected to the components
- Modelling the Caldicot Levels' flood defence system and expressing this model into data
- Calculation of the probability of flooding of the Caldicot Levels' flood defence system

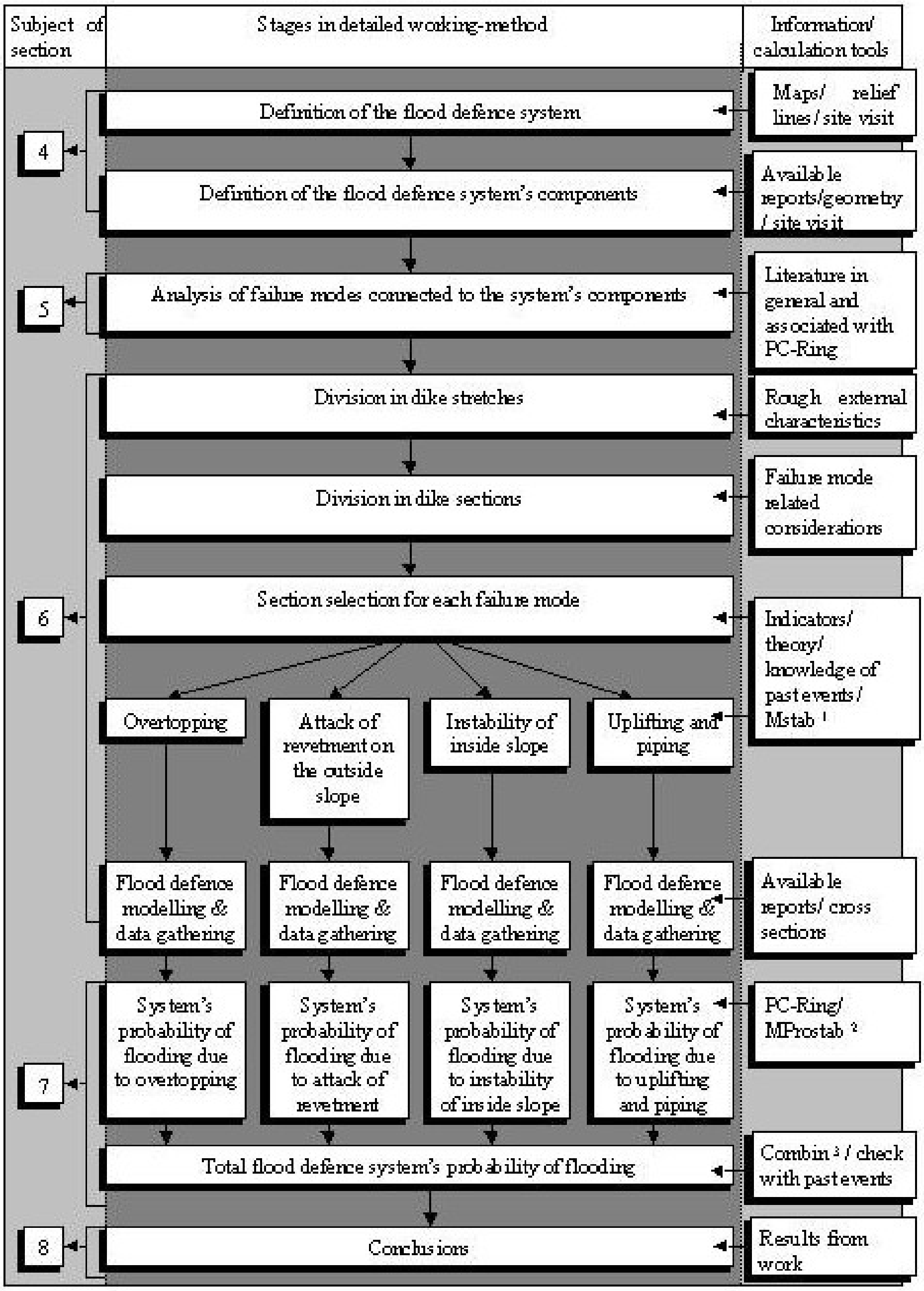
Two scenarios are subject of the reliability analysis:

1. The Caldicot Levels' flood defence system as it is at present
2. The same system after a number of currently planned improvements have been taken into account

Before the reliability analysis is discussed in more detail an overview is given of PC-Ring, the reliability method which is used to make the calculations, and a description is given of the hydraulic climate along the Caldicot Levels' flood defence system.

2 Reliability-based flood defence design tools in the Netherlands

The reliability-based flood defence tools as applied in the Netherlands mainly consist of software called PC-Ring, see sections 2.1 to 2.3 and Vrouwenvelder et al. (1999 and 2001) for details. This software is used to calculate the total annual probability of failure of a flood defence system consisting of dikes. However, before these calculations can be made the actual flood defence system must be translated into a model. This model is expressed into data and these data can be used in the calculations with PC-Ring. The amount of work related to data gathering is reduced by a modelling process which aims to select the sections that are most representative of the system's probability of failure. This process starts with dividing the flood defence system into stretches, and more detailed sections. The cross sectional and statistical properties are assumed to be constant along one section. By use of indicators, which are based on rough information, sections are selected which are considered as weak. These selected cross sections dominate the total probability of failure and are therefore included in the calculations with PC-Ring. This process is described in Calle et al. (2001).



[1] *Mstab calculates the stability factor according to Bishop in relation to instability of the inside slope.*

[2] *MProstab calculates the probability of failure due to instability of the inside slope given a certain water level.*

[3] *Combin is a part of PC-Ring that combines the probabilities of failure due to the different failure modes to one total probability of failure.*

Figure 1: Detailed outline of reliability analysis

Combining the steps as mentioned in section 1 with this process provides the outline of the reliability analysis as presented in Figure 1.

2.1 *Failure modes in PC-Ring*

Below the failure modes are mentioned that are included in PC-Ring (see Vrouwenvelder et al., 2001), in Figure 2 is shown how these failure modes relate.

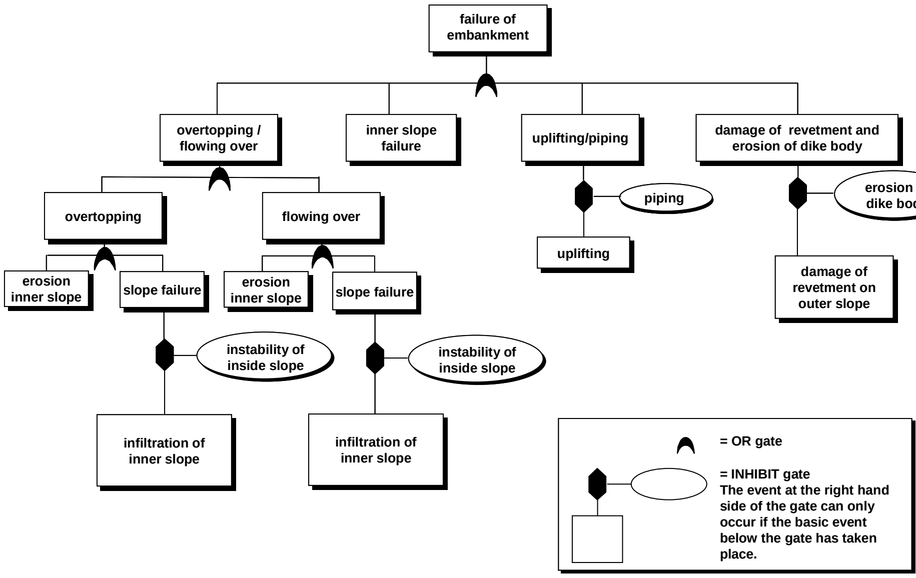


Figure 2: Failure modes and their mutual relations in PC-Ring. Only the structural failure part is implemented in PC-Ring

- Overtopping/ overflow
- Instability of the inside slope
- Uplifting/ piping
- Attack of the revetment on the outside slope

2.1.1 Overtopping/ overflow

Overtopping or overflow discharges pass the dike crest and consequently failure occurs either due to damage and erosion of the inside slope or due to saturation of the clay cover layer soil leading to instability of the inside slope, see Figure 3. The limit state function which is implemented in PC-Ring in relation to failure due to overtopping is:

$$Z = m_{qc}\, q_c - m_{qo}\, q_o \;/\; P_t$$

In which q_c is the critical discharge expressing the limit discharge for which almost damage of the grass occurs, q_o is the actual occurring overtopping discharge due to the hydraulic boundary conditions in combination with the geometry of the dike, m_{qc} is the model uncertainty with respect to the critical discharge q_c, m_{qo} is the model uncertainty with respect to the actual discharge and P_t is the part of time that overtopping occurs, this variable is applied to take the pulsating character of overtopping in account.

In case of discharges due to overflow the limit state function is expressed by:

$$Z = h_{kd} - h = h_d + \Delta h_c - h = h_d + \sqrt[3]{\frac{2.78 \cdot q_c^2}{g}} - h$$

In which h_d is the crest level of the dike, h_c expresses the critical height for which almost damage of the grass occurs and h is the actual occurring water level.

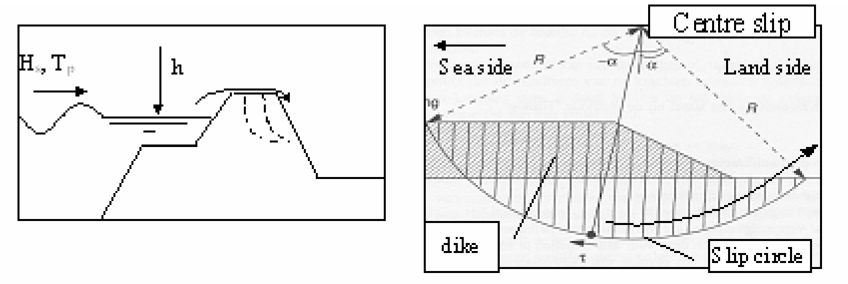


Figure 3: Failure of the dike due to overtopping/overflow discharges (left). Failure of the dike due to instability of the inside slope according to the method of Bishop (right).

2.1.2 Instability of the inside slope

The extreme outside water levels result in different water pressure distributions in the dike body. The geotechnical equilibrium (according to Bishop) of the ground body is affected in such a way that instability of the inside slope occurs, see Figure 3. Software called MProstab is available to calculate the probability of failure due to instability of the inside slope given a certain outside water level. The following limit state function is applied:

$$Z = \Gamma - q$$

In which Γ is the stability factor according to Bishop and q is the threshold value of the stability factor for which instability occurs, this value is usually 1. The β and α values given three different water levels which result from the MProstab calculations are applied in the limit state function in PC-Ring:

$$Z = \beta(h) + \sum_{i=1}^{n_{MPROSTAB}} \alpha_i(h)\, u_i$$

In which $\beta(h)$ is the reliability index ($=-M^{-1}(p_f)$, with M the cumulative function of a standard normal distribution) given the water level resulting from the MProstab calculations, $\alpha_i(h)$ are the influence coefficients given this water level and u_i are variables with a standard normal distribution.

2.1.3 Uplifting / piping

The hydraulic uplifting force exerted by the water head difference between the outside and inside water levels leads first to bursting of the impervious foundation layer of the dike. After uplifting of the impervious layer, water flow as a result of the hydraulic head difference causes the development of pipe shaped erosion in the burst impervious layer. This failure mode is represented by two limit state functions, one describing the process of uplifting:

$$Z = m_o h_c - m_h (h - h_b)$$

In which h_b is the water level "inside", h is the water level outside and h_c is the critical water level or the limit water level for which almost uplifting occurs, see also Figure 4. h_c is determined with a model based on the properties of the impervious layer. m_o takes the model uncertainty of the model which determines h_c in account and m_h the level of damping.

The other limit state function describes the process of piping:

$$Z = m_p h_p - (h - 0.3d - h_b)$$

The dike fails as a consequence of piping if the difference between the local water level h and the inside water level h_b, reduced with a part of the vertical seepage length d, exceeds the critical water level h_p. m_p is the model uncertainty of the model with which h_p is described. The critical water level h_p is described by Sellmeijer's model of piping.

2.1.4 Attack of the revetment on the outside slope

Attack of the revetment on the outside slope by the water and wave conditions causes damage to the revetment. The dike body is exposed to the same hydraulic conditions after the revetment has been damaged. Erosion of the dike body can lead to breach, see Figure 4. Each type of revetment fails differently due to loading by the waves and therefore requires a different limit state function. The limit state function describing the process of erosion of the dike body is equal in each situation. In PC-Ring three main different types of revetment are discerned: Grass, riprap, asphalt. The description in this paper is limited to the limit state functions in connection to grass. More detailed information about the other revetment types can be found in Steenbergen et al. (2004) and Vrouwenvelder et al. (2001). In case of grass, the process of damage to the revetment and the process of erosion of the dike body are integrated in one limit state function:

$$Z = t_{RT} + t_{RK} + t_{RB} - t_s$$

In which t_{RT} is the time that a storm takes to damage the grass, t_{RK} is the time that a storm takes to erode the clay cover layer and t_{RB} is the time that a storm takes to erode the rest of the dike body. t_s represents the duration of the storm.

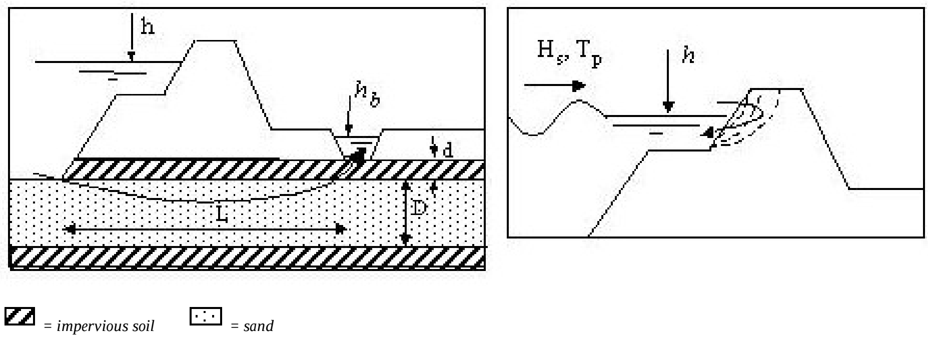


Figure 4: Failure due to uplifting and consequently piping (left). Failure due to damage of the revetment on the outside slope and consequently erosion of the dike body (right).

*2.2 **Statistics in PC-Ring***

The statistical character of the random variables in the limit state functions consists of statistical distribution functions and correlations in time and space. The correlation in time is modelled according to Ferry Borges and Castanheta which assumes constant correlations during and between time intervals. The spatial correlation of random variables is assumed to decrease from full correlation to a limiting value. The loading of the dikes is introduced by different types of hydraulic climates: lakes, rivers, coasts. In case of for instance a tidal river, the statistical models of the water levels and wind speeds involves a set of basic random variables of water levels and wind speeds at the mouth of the river, see Figure 5. These basic variables are transferred to local water levels and wave conditions by use of a numerical model like for instance Mike11, Sobek, SWAN, HISWA, DELFT 3D (see for instance Postma et al). The statistical distribution functions and the correlations between water level and wind speed are applied to the basic random variables at the mouth of the river. Detailed information can be found in Vrouwenvelder et al. (2001).

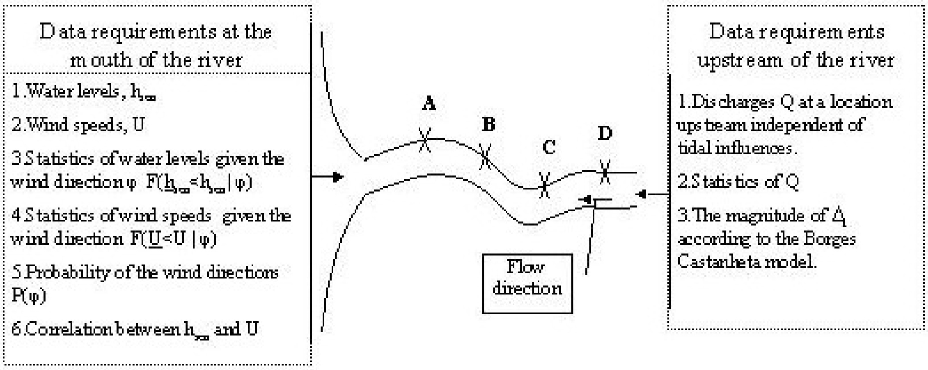


Figure 5: Data requirements with respect to the statistical model of the hydraulic climate at a tidal river

2.3 Probabilistic calculation methods in PC-Ring

Calculations at the level of a single limit state function can be made with: FORM, SORM, crude Monte Carlo, Directional Sampling. Detailed information about these methods can be found in Vrouwenvelder (2001) and Vrijling and Van Gelder (2002). In Vrouwenvelder et al. (2001) methods are presented to calculate an annual probability of failure of a flood defence system involving a number of different limit state functions representing: one failure mode, cross section, tide, wind direction. These different limit state functions are combined to one remaining equivalent limit state function. During this process mutual correlations between the functions are taken into account.

3 Hydraulic climate along the Caldicot Levels' flood defence system

The Caldicot Levels' flood defence system is located at the south coast of Wales in the UK. The system borders the Severn Estuary in the south, the river Usk in the west and a distinct line of hills in the north and east, see Figure 6. ABP Research (2000) provides information on the local water level, wind speed and wave conditions. In the Severn estuary one of the largest tidal ranges in the world occurs, varying between 9 and 15 meter. The largest fetches and the most severe wind speeds are related to the southwesterly wind directions. The River Usk is a small river, however the water levels can reach relatively high values especially in case of high water levels at the Severn Estuary. The mean elevation of the Caldicot Levels is OD+5.5m which compares to mean tide high water levels of OD+4.8m, mean spring tide high water levels of OD+6.5m and a 200-year return period water level of OD+8.55m. The crest levels of the flood defence system vary between OD+8 and OD+10m. According to WS Atkins (1999), the dikes and soil underneath consist mainly of clay. Finally, in WS Atkins (2000) information is available of areas which have been subjected to damage caused by a number of storms in the past and which can be considered as weak.

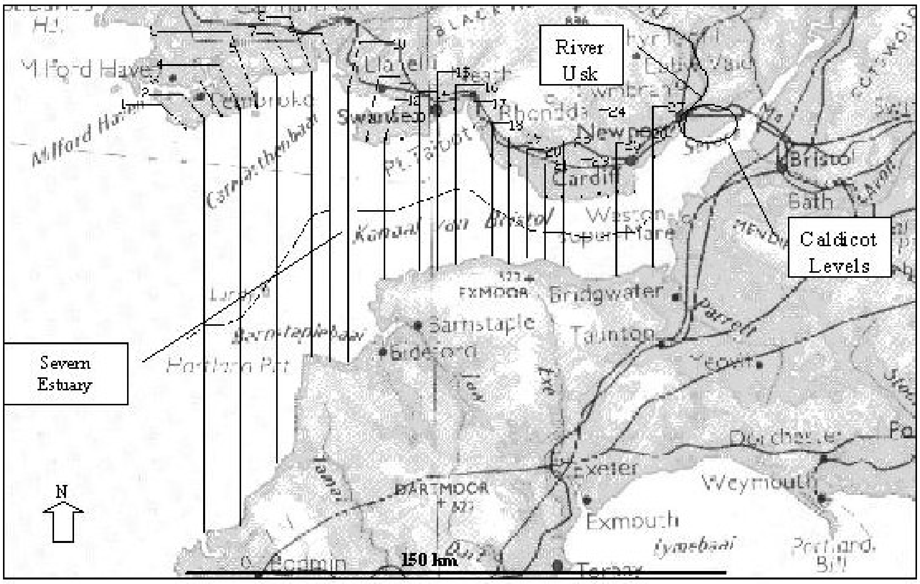
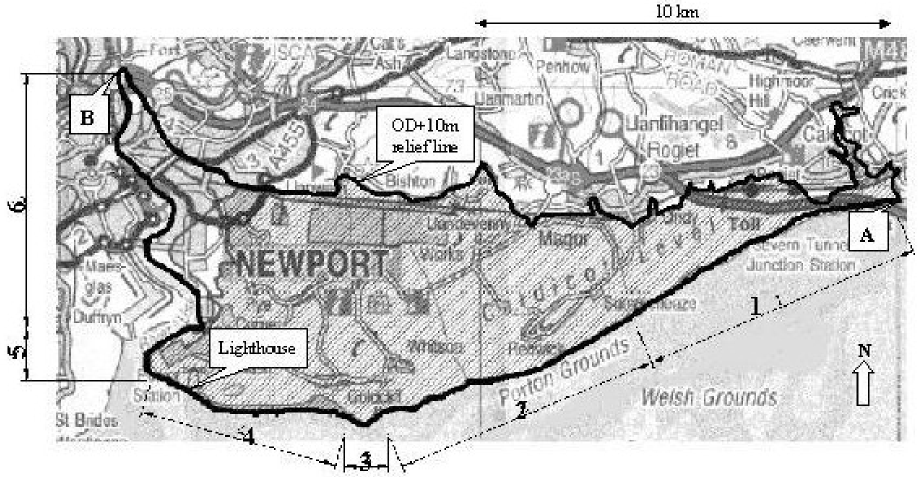


Figure 6: Location of the Caldicot Levels' flood defence system with respect to the Severn Estuary and the River Usk



1= Dike

2= Dike with wave return wall

3= High grounds with masonry wall facing

4= Raised grounds along Severn Estuary

5= Raised grounds along the Usk

6= River banks of the Usk

Figure 7: Definition of the Caldicot Levels'flood defence system's boundaries. The OD+10m line represents the high grounds which do not contribute to the system's probability of flooding. Additionally, the figure includes a rough indication of the position of the main flood defence components

4 Definition of the flood defence system and its components

The Caldicot Levels' flood defence system is defined as shown in Figure 7 (original figure from Chatterton (2001)). The line south of the locations marked A and B represents the relevant defence length for the calculation of the probability of inundation. The OD+10m relief line defines the boundary formed by the high grounds. The area between the lines suffers consequences in the form of partial or complete flooding if the flood defence system fails at one or more locations. The components are also indicated in Figure 7.

5 Analysis of the failure modes connected to the components

5.1 ***Failure modes of the present flood defence system***

In case of the calculation of the probability of failure of the present flood defence system all of the components are calculated with the failure modes as present in PC-Ring, see subsections 2.1.1 to 2.1.4. Below the main flood defence components which are also mentioned in Figure 7 are listed with the failure modes which have been considered in the calculations:

- Dike: overtopping, instability of the inside slope, attack of the revetment on the outside slope (grass). The contribution of failure due to piping is considered to be negligible as the seepage lengths are very large.
- Dike with wave return wall: overtopping, instability of the inside slope, attack of the revetment on the outside slope (rock armour). In case of the present flood defence system the effect of the wave return wall on the overtopping is assumed to be negligible. The approach of the wave return wall in case of the improved system is described in subsection 5.2 The contribution of failure due to piping is considered to be negligible as the seepage lengths are very large.
- High grounds with masonry wall facing: the high grounds are regarded as broad dikes with shallow slopes. Therefore, the contributions to the total probability of flooding by the failure modes instability inside slope, piping, attack of the revetment on the outside slope are assumed to be small. Breach of the broad dike due to overtopping and erosion is also assumed to be unlikely. However, the overtopping discharges might lead to considerable damage to assets on or directly behind the dike. Thus, only (non-structural) failure due to overtopping is taken into account and approached with a self-chosen limit critical discharge value instead of the grass / erosion or saturation models.
- Raised grounds along the Severn Estuary are approached in a similar way as high grounds with masonry wall facing.

- Raised grounds along the River Usk are approached in a similar way as raised grounds along the Severn Estuary.
- The river banks of the Usk: failure is represented by the return period of the river water level exceeding the highest elevation of the river bank.

5.2 *Failure modes of the improved flood defence system*

The main flood defence components will be improved in future. These improvements concern mainly raising of the flood defences along a considerable length and the replacement of the present wave return wall by a higher and more effective one. For all the flood defence components except the dike with wave return wall the selection of failure modes remains the same for the present and the improved system. For the improved form of the dike with wave return wall an approach is developed which takes the wave return wall into account.

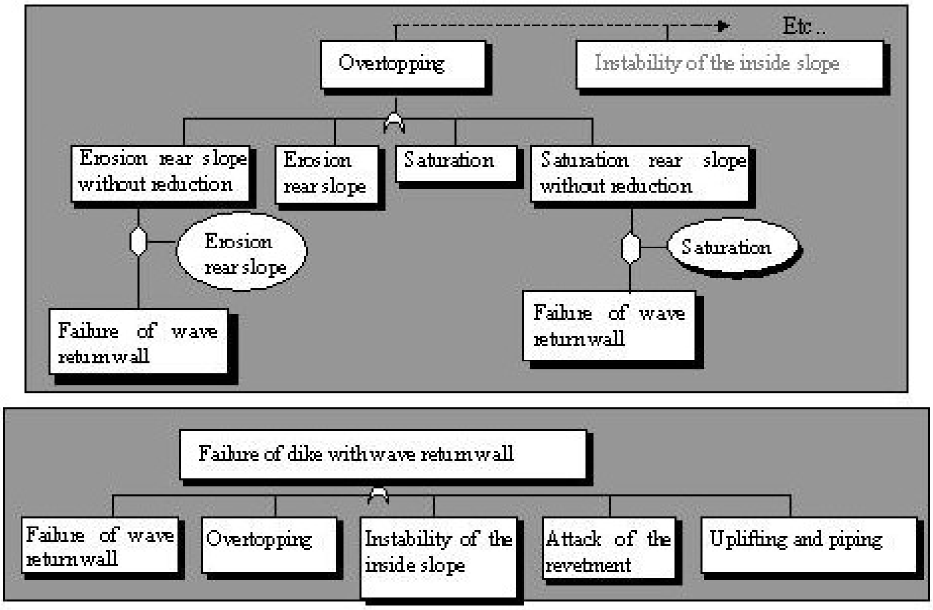


Figure 8: Fault tree of theoretical approach of wave return wall (upper part) and of practical approach as implemented in PC-Ring (lower part)

5.2.1 *Relation between failure of the wave return wall and of the dike*

Theoretically speaking, the wave return wall reduces the amount of overtopping and failure of the wave return wall does not necessarily need to lead to failure of the complete dike (upper part of Figure 8). However, this approach leads to practical complications of the implementation of the wave return wall in PC-Ring. Therefore, the practical approach as illustrated in the lower part of

Figure 8 is applied. This approach is based on the assumption that when the wave return wall fails, the entire dike also fails. Analysis of the results of the calculations will point out whether the probability of failure of the wave return wall is high or low relative to that of the dike. If this is high, the probability of failure of the dike dominates the probability of failure. If it is low the practical approach applies, the probability of failure is dominated by the highest of the following two values: probability of failure of the wave return wall or the probability of failure of the dike with wave return wall as a whole.

5.2.2 Failure modes of the wave return wall

Two failure modes of the wave return wall are taken into account: horizontal sliding or tilting. The force is formed by the wave impact and the strength is mainly determined by the weight of the wave return wall. The limit state function representing horizontal sliding is implemented as follows:

$$Z = {}^{2}/_{3} \tan(\varphi)V - H$$

The resulting wave impact horizontal forces H exceed the friction force as a result of the weight of the wave return wall. The model which is applied to determine the wave impact pressures is described in Martin et al. (1999) and Kortenhaus et al. (2000), see Figure 9. V is the resulting vertical weight, the friction coefficient is expressed by $2/3\tan(\varphi)$, in which φ is the effective angle of internal friction of the soil.

The limit state function of failure due to tilting of the wave return wall is expressed by:

$$Z = {}^{1}/_{6}b_f - {}^{M}/_{V}$$

The wave return wall fails due to tilting if the resulting force M/V is not within the core of the foundation plane. M is the resulting moment of the horizontal wave impact forces with respect to the centre of the foundation plane, for V see sliding, bf is the width of the foundation plane.

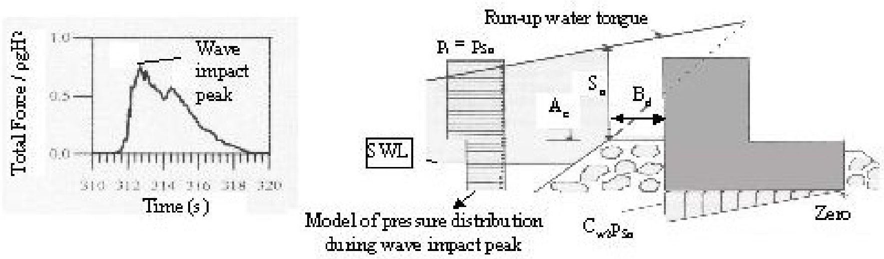


Figure 9: Model of wave impact on crown walls as applied in the sliding and tilting limit state functions of the wave return wall (from Martin et al. (1999))

6 Modelling the flood defence system and expressing this model into data

As is mentioned in section 2, the Caldicot Levels' flood defence system is divided into stretches and more detailed sections. For each failure mode the relevancy has been determined and the weak sections have been selected according to rough indicators. The after the selection remaining sections represent the model of the flood defence system which is applied in the probabilistic calculations. The main obstacles in the data requirements prove to be the statistical models of the hydraulic boundary conditions, the numerical models of the local water levels and the general statistical data availability. In case of the Severn Estuary a model of the local water levels has been set up with Mike11 based on limited information with respect to: geometry of the estuary and the actual occurring water levels which can serve to calibrate and validate the model. Part of the applied network can be found in Figure 6. Moreover, information about wind speed and water level statistics did not appear in the for PC-Ring required form. In case of the River Usk a numerical model of the local water levels is available in Mike11, though only limited discharge statistics were available.

7 Calculated annual probability of flooding, present and improved system

The calculations of the annual probability of flooding of the Caldicot Levels' flood defence system result in:

- Annual probabilities of failure and reliability indices β of the selected sections, Figure 10.
- Coefficients of influence, or α-values.
- The total annual system's probability of flooding due to one failure mode and the accompanying weakest link in the system see table 2.
- The total annual system's probability of flooding, see table 2.

The dominating failure mode turns out to be overtopping (see Table 2). The weak areas which result from the calculations correspond with the more severely attacked areas in the past storms. The coefficients of influence point out that the uncertainty associated with in the first place the water levels and in the second place the wind speed and wind direction, contribute most to the total probability of failure. From this information the main reasons causing the above mentioned areas to be weak are derived: a low crest level in combination with the orientation of the dike with respect to the south westerly wind directions. These wind directions are related to high wind speeds and large fetches and are therefore associated with high levels of wind set up and more severe local wave conditions.

Table 2: Results from the calculations of the annual probability of flooding of the Caldicot Levels' flood defence system in its present and improved form

		Overtopping			Instability of the inside slope			Attack of the revetment on the outside slope			Total system		
		Weakest link No.	**β**	β_o	**Weakest link No.**	**β**	β_i	**Weakest link No.**	**β**	β_r	**Weakest link No.**	β_{tot}	P_f
Pre-sent	With Usk	68	0.64	0.51	79	1.2	1.2	14	3.1	3.1	68	0.29	0.39
	Without Usk	28	1.09	0.91							28	0.59	0.28
Impr oved	With Usk	68	0.64	0.51							68	0.53	0.30
With-out Usk	Failure wrw	31	1.08	1.02									
	Failure dike with wrw	63	1.75	1.67									
	Failure dike without wrw	63	1.75	1.67	46	3.7	3.7	29	4.5	4.4	63	1.6	0.06

wrw = wave return wall. β_o: reliability index for overtopping,
β_i: reliability index for instability, β_r: reliability index revetment

The annual probability of failure of the wave return wall is determined by failure due to tilting and is relatively high (see Table 2). Because of this high probability, the assumption that the complete dike fails if the wave return wall fails is not justified in this case. Therefore, based on these results the actual probability of failure is expected to be a combination between failure of the dike with the influence of the wave return wall on wave overtopping and failure of the dike without a wave return wall on the crest (see Figure 8, upper part). In Figure 10 the former scenario is referred as "improved, no failure w.r.w.", whilst the latter scenario is referred as "improved, no w.r.w. present on crest". With respect to the magnitude of the probability of failure of the wave return wall must be noted that the dynamic nature of the wave impact pressures (Figure 9) is neglected in the sliding and tilting limit state functions. This may have a heightening effect on the probability of failure of the wave return wall.
As overtopping is the dominating failure mode, in Figure 10 the reliability indices in connection to failure due to overtopping of the selected sections are given for the present and improved flood defence system. Figure 10 points out that the planned improvements are unbalanced:

- Sections 1 to 20 are much improved compared to the sections with wave return wall, 21 to 40, 43 to 57, 60 and 62. The latter mentioned sections are moderately improved, moreover their reliability indices are very irregular.
- Table 2 points out that the sections along the Usk provide the weakest link. These sections are not improved at all.

- Without considering the sections of the river Usk, section 63 turns out to be the weakest link: this is one of the sections for which no improvement is planned.

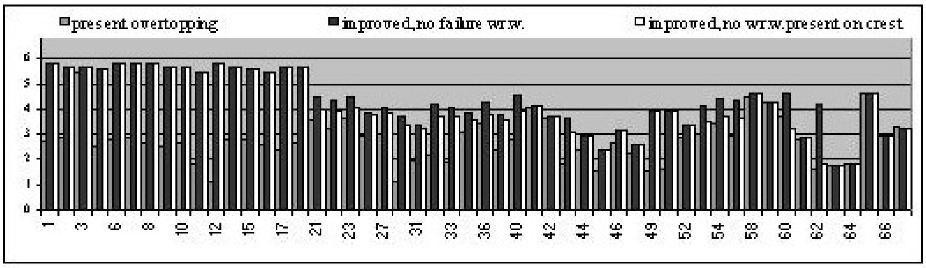


Figure 10: Reliability indices of failure due to overtopping of the present flood defence system and the improved flood defence system. For the improved flood defence system the reliability indices of dikes with and without the influence of the wave return wall on the overtopping discharges are included. Below the flood defence components and the corresponding section numbers are given.

Compartment no. 1t/m 19 and no. 49 t/m 52	*= dike without additional structures*
Compartment no. 20 t/m 48, no. 53 t/m 57	*= dike with wave return wall*
Compartment no. 41 and 42	*= high grounds with masonry wall facing*
Compartment no. 58 t/m 62	*= raised grounds along the Severn Estuary*
Compartment no. 63 t/m 67	*= raised grounds along the river Usk*
Compartment no. 68 t/m 78	*= Usk river banks (not included in the plot)*

8 Conclusions

The study has demonstrated how reliability analysis of a flood defence system identifies defence sections and system components that make the greatest contribution to flood risk. This information can then be used to target inspection, maintenance and upgrade activities. The combination of probabilistic analysis of the flood defence system with quantified analysis of potential impacts of flooding in the zone protected by the defences provides a quantified estimate of flood risk, which can be used to justify and optimise economic investment in flood defence improvements. In order to apply the Dutch flood defence design methodology to the UK, a new failure mode relating to failure of the wave return wall had to be introduced into the PC-Ring program. Moreover, as is often the case for complex failure of flood defence dikes, this new mechanism interacts with other mechanisms during the failure process. Combining separate mechanisms with logical OR-gates is a simplification. However, the mathematical relations in PC-Ring are suitable to include the additional failure modes in the calculations. Furthermore, even though the Caldicot site was relatively well provided with data for UK standards, there was still not all of the data that would ideally be necessary for application of PC-Ring.

The study has highlighted some of the differences between flood defence systems in the UK and the Netherlands (see Table 3 and also Hall et al. 2000) and the implications for quantified analysis. There are 35,000 km of flood defences in the UK ranging from low earth embankments protecting just a few fields to the Thames Barrier protecting central London. Because of this diversity, a range of appropriate methods are required for risk analysis that are suited to the potential severity of the consequences of flooding and the available information. All of these methods should be risk-based in some sense, the intention being that risk should form the basis of decision-making at all levels in the UK, from national policy decisions, regional strategic plans, to project specific appraisal, design, operation and maintenance decisions. The detailed reliability methods being promoted in the Netherlands and implemented in PC-Ring are best suited to the design and appraisal of, by UK standards, relatively large and highly engineered flood defence systems. Even for these important systems, the study described in this paper has demonstrated that some adaptation of the Dutch reliability methods and additional data collection is inevitable. Recommendations with respect to possible adaptations of the Dutch methods following from this study concerned among others whether a “tailor-made” set up, such as in PC-Ring, or a “one-size-fits-all” set up of reliability software is desired in the UK. The tailor-made set up is based on implementing all possible failure modes in the program code, whereas the one-size-fits-all set up is based on flexibly entering the limit state functions, statistical data and the desired mathematical relations between the limit state functions. A number of typical systems can be set up default in the program, but are easier to adjust because of the flexible nature of the program.

Table 3: Comparison of issues surrounding the introduction of risk-based coastal engineering and management in the UK and the Netherlands (from Hall et al. (2000))

UK	**Netherlands**
Very diverse risks, with some large urban and agricultural areas at risk but also very many small flood risk areas.	Very large polders with devastating consequences of failure.
Tradition of permissive legislation on flood and coastal defence.	Tradition of prescriptive legislation on flood and coastal defence.
Primarily economic decision criteria.	Decisions primarily based on legal safety standards.
Decentralised engineering and decision making.	Centralised engineering expertise
Some reservations about current safety levels.	General acceptance of the current safety level.

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