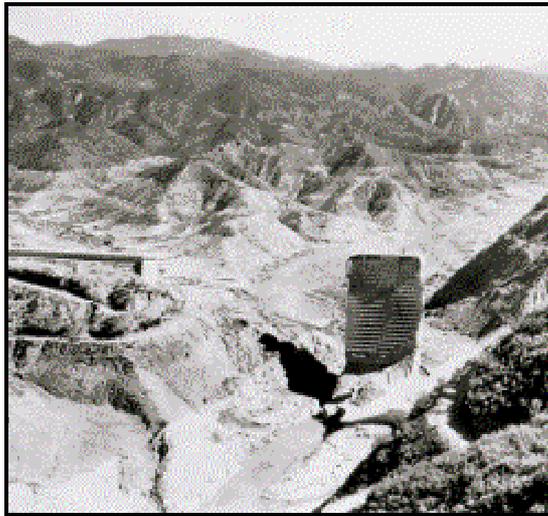


Investigation of Extreme Flood Processes & Uncertainty (IMPACT)

Identifying Potential Breach Location



WP2: Deliverable D2.4.1

December 2004

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1. Introduction

Earth embankments form a critical component of many EC countries flood defence systems. In comparison to other flood defence types (e.g. walls, piling etc), embankments typically form the greatest percentage of defence type. In the UK alone, it is estimated that there are approximately 35000km of flood defence embankment – all of which require regular inspection and maintenance to ensure that flood defence performance is maintained at an acceptable and appropriate level.

With such a maintenance commitment, any systemised approach that will allow maintenance works to focus on priority areas of embankment will allow more effective use of limited resources. As specified in the DoW, *the objective of this work package is to try and identify whether the likely mode and location of a breach failure in a long length of embankment can be established – in relative risk terms. This would allow owners of large lengths of flood embankment to prioritise maintenance works according to the risk of breaching.*

This report addresses IMPACT Deliverable D2.4.1 - Breach location methodology / model to identify the relative risk of breach formation in long lengths of flood defence embankment.

2. Existing methodologies and potential approaches

2.1. Generic approaches

Before considering what approaches are being developed in Germany, Netherlands and the UK, it is worth considering some generic issues. When looking at breach location, two scales of analysis may be considered:

- Local: What local factors are affecting embankment performance
- System: How can you represent embankment performance within a system of defences, and can relative risk of failure be identified?

Each approach is relevant. The system approach potentially allows the relative risk of failure of defence lengths to be considered, and hence could permit focussing of attention down to critical lengths. The local approach allows consideration of a defence length, and understanding of specific local issues, so permitting effective repair or maintenance works. In this context, a defence length could comprise a section of defence embankment ranging from metres to kilometres in length.

Work undertaken in WP6 addresses the issues of local factors. IMPACT partner H-EURAqua has been collating historic records of embankment failures in Hungary and Czech Republic for the purpose of analysis to identify typical factors affecting breach. The focus of this report therefore addresses the system approach to identifying breach location although an overview of potential approaches is reviewed in Section 4.

2.2. UK, Dutch and German approaches...

Development of a systems based approach for identifying flood risk (and hence the performance of flood defence structures, hence the risk of breach formation through defence embankments) is an area where significant progress has been made during the last few years. The countries most active in developing these system risk based approaches are the UK, Netherlands and Germany.

In the UK the Environment Agency / Defra joint research programme on flood defence continues to fund a series of research development projects, building on an original project called RASP: Risk Assessment of flood and coastal defences for Strategic Planning. This project aimed to develop tiered methodologies for the risk assessment of flood defence system performance. RASP used and built upon reliability-based design tools developed for dike rings in the Netherlands.

The tool used in the Netherlands is software called PC-Ring which has been developed to allow system based calculations on the risk of flood defence failure. 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.

Sea dikes are amongst the most important coastal structures along German coastlines. Reliability and risk based design concepts have been increasingly proposed during the last years in Germany (see Oumeraci and Kortenhaus, 2002). The probabilistic methods on which these concepts are based allow accounting for the uncertainties in the input parameters and the models describing possible failure modes of various types of coastal structures. However, these methods are very often limited to simple cases or to just one or a couple of failure modes.

2.3. An introduction to PC-Ring

Technological developments of computers in the nineties have enabled the calculation of probabilities of failure of flood defence systems. The limit state functions applied in this software are based on former non-probabilistic reliability methods employed in the Netherlands.

The probabilistic calculations done with PC-Ring correspond with the detailed level methodology in RASP and take into account a large number of flood defence sections and multiple limit state functions. The main outputs from these calculations are:

- Probabilities of failure for each flood defence section included in the calculations and therefore an overview of the weak areas in the flood defence system;
- Insight into the key failure mode of a system of flood defences;
- Insight into the uncertainties associated with the parameters in the limit state functions and their contributions to the total probability of failure of the flood defence sections.

The local probabilities of failure can be used in determining the spatial distribution of risk. Defence improvements can be targeted to flood defence sections contributing most to the risk in the floodplain. The insight in which model parameter contributes most to the probability of failure of the flood defence section in combination with the limit state function provides advice on what kind of improvement activities are expected to be most effective. The effectiveness of different improvement options can be compared by analysing the reduction of risk in the floodplain associated with each option (see Buijs et al. (2003)ⁱ).

The reliability models applied in the limit state functions in PC-Ring are similar to those applied in the UK within RASP. The limit state functions of the failure modes implemented in PC-Ring are given as an example of how to fit current practice models for failure modes into limit state functions. Specific details can be found in Buijs (2003)ⁱⁱ and translated from Vrouwenvelder et al. (2001) and in Lassing (2003)ⁱⁱⁱ. A very comprehensive discussion of limit state functions in connection to embankments can also be found in Kortenhuis & Oumeraci (2002)^{iv}.

2.3.1. An introduction to the failure modes used in PC-Ring

PC-Ring only includes failure modes concerned with structural failure. The following failure modes of embankments are included in PC-Ring:

- Overtopping/overflow.
- Instability of the inside slope.
- Piping.
- Damage of the revetment on the outside slope and consequently erosion of the embankment body.

The fault tree of these failure modes related to an embankment is given in Figure 1.

To calculate the likelihood of failure via each of these different failure modes, limit state equations have been developed and are applied in conjunction with a measure of loading. This approach allows the analysis of an entire flood defence system and permits extraction of the most probable cause and location of failure. A more detailed explanation of this approach can be found in Appendix A, including details of the limit state conditions.

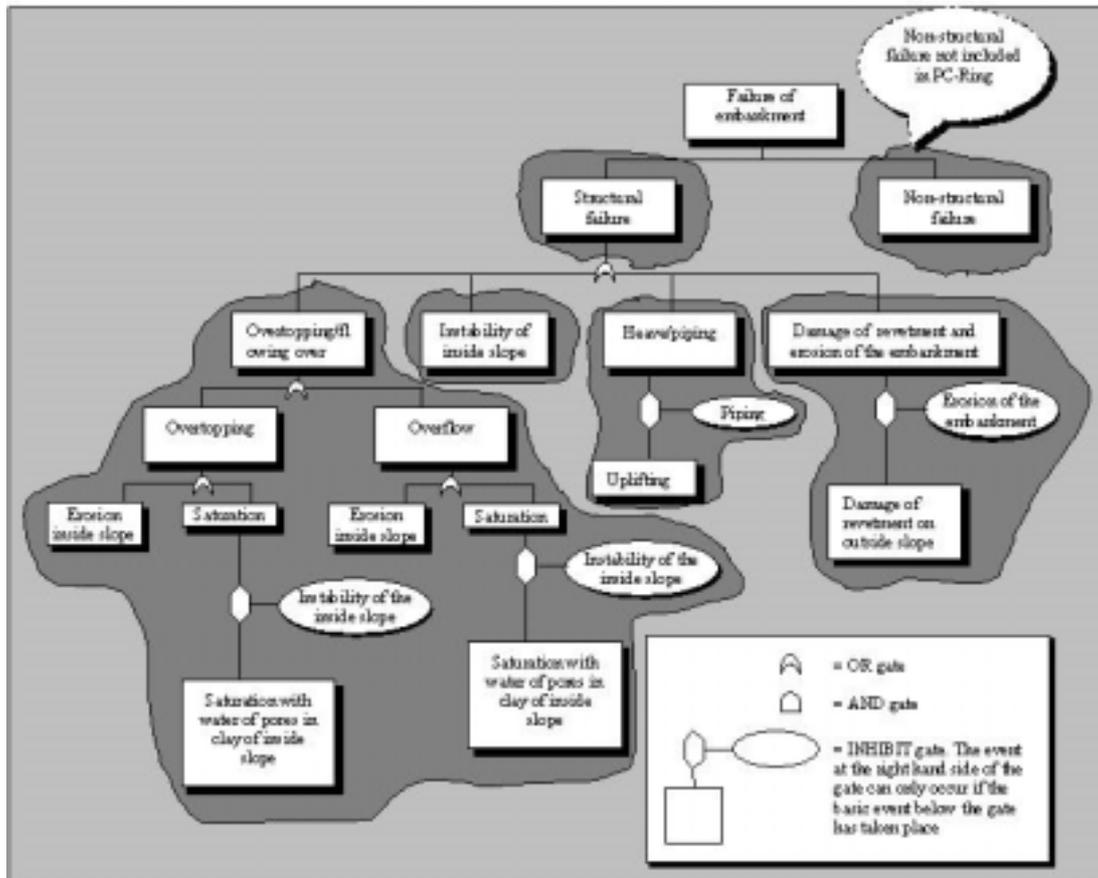


Figure 1 Failure modes of the embankment as applied in PC-Ring

2.3.2. Main steps taken in PC-Ring calculations

The following steps are taken in the calculation of the flood defence system's probability of flooding:

1. Calculation of the probability of failure of one flood defence cross section for one tide, one partial failure mode (for instance failure mode overtopping, partial failure mode saturation), given the wind direction.
2. Combination of the partial failure modes resulting in the probability of failure of one total failure mode.
3. Taking the probability of the wind directions into account.
4. Determining the probability of failure due to one failure mode for the total flood defence stretch for which the under step 1 mentioned flood defence cross section is representative.
5. Combining the probabilities of failure of all the wind directions.
6. Determining the probability of failure for the total regarded period.
7. Combining the probabilities of the different failure modes.
8. Combining all the flood defence stretches to find a total flood defence system's probability of flooding.

The process of cross section selection and data gathering is as follows:

1. Division of the water defence system in *defence types*.
2. The next step is to divide the water defence system in *embankment stretches*. To this end rough information and insights are used. The division is based on the external physical characteristics and not yet on the characteristics directly connected to the failure modes, although implicitly the connections are there. The following characteristics are important for this selection^v:

- Orientation to the wind directions: embankment stretches that are orientated differently will be loaded by different wave regimes and therefore must be discerned as different embankment stretches.
 - High water regimes, differences in extreme water levels: lengths of the water defence system for which different high water regimes are relevant must be discerned as separate embankment stretches.
 - Geometrical characteristics foreshore: Embankment stretches with significant different sizes of the foreshore in terms of height and width.
 - External geometry of the water defence: lengths of the water defence system with significant differences in height and (external) construction are discerned as different embankment stretches. Differences in geometry will lead to differences in loading conditions due to the same hydraulic boundary conditions.
3. The water defence system has now been divided into rough stretches with the same type of characteristics. However, cross sections have to be determined which can be regarded as representative of the total embankment stretch. If still significant differences between cross sections in an embankment stretch are present, then the stretches need to be divided into further parts: *the flood defence sections*. A first insight with regard to weak spots in the water defence system can be given by managing authorities. These authorities can also help to determine the relevant failure modes for certain embankment sections. The following failure mode related considerations can form a basis for a division in embankment sections:
- Different types of outside slope revetment (types and construction), this will lead to a further division for the failure mode: damage of the revetment on the outside slope and erosion of the embankment body.
 - Differences in geometry on a detailed level have consequences for each of the failure modes.
 - Differences in the foundation soil can have a considerable effect on the contributions to the probability of failure of geotechnical failure modes. For these failure modes a further division has to be made.
 - Differences in the inside slope revetment of embankments (quality of the grass, thickness and qualification of the clay cover layer on the inside slope, the angle of the inside slope, etc.), this kind of information is especially useful for the division in embankment sections for the failure modes overtopping and consequently erosion of the inside slope or instability of the inside slope.
 - Information about the construction of the embankment (clay embankment, sand embankment with a clay core, etc...) and information about the soil layers underneath and directly next to the embankment (in front of and behind the embankment), this information is relevant to the failure modes heave and piping, instability of the inside slope and damage to the revetment and erosion of the embankment body.
4. After step 3 the water defence system has been divided in embankment sections. The following step is to *select the relevant sections* for the reliability analysis. The procedure to come to this selection of sections is given below:
- Regard a failure mode for which it is desirable to reduce the number of embankment sections. The first logical step is to eliminate all the embankment sections for which the mode is not relevant, or in other words: the contribution of the section to the probability of failure due to a certain mode is negligible in advance. Well known weak spots can provide valuable first insight in which sections contribute significantly and which ones do not.
 - For the remaining sections, indicators are used to rank them. These indicators are related to the failure modes. The number of selected sections can be limited based on this ranking.
 - Apart from the selected weak sections based on the indicators, sections from the middle and strong categories have to be chosen. This is done because it is practical to

- be able to make an estimate of the probability of flooding after eliminating the weak spots in the probability of failure calculations.
- The former three steps have to be performed for all failure modes, which can result in a different selection of embankment sections for each mode.
 - Check the spreading of the sections along the water defence system with respect to the magnitude of the expected consequences. The sections with substantial consequences that fall out off the analysis should be added or shift a bit with the choice in the middle and strong sections. With the total risk analysis in mind, a strong section with extensive consequences can contribute just as much or even more to the total risk as a weak section with hardly any consequences.
5. The first selection of embankment sections has been finished. For each section it is clear which failure modes are regarded. The next step is *to gather data* for the regarded failure modes and embankment sections.
 6. After the first calculations of the probability of failure with PC-Ring a check is made if sections have to be added or adjusted:
 - First the contribution of the failure modes to the total probability of failure is considered.
 - For the mode with the largest contribution a check has to be made if the last selected sections still have a significant contribution to the probability of failure. If they do an additional selection of sections for that mechanism has to be made. This check needs to be made for all the failure modes with an emphasis on the ones with the largest contribution.
 - This process has to be repeated until no more sections need to be included in the analysis. The number of cycles depends on the number of embankment sections which are chosen initially and after expansion.

These steps are shown schematically in Figure 2 below.

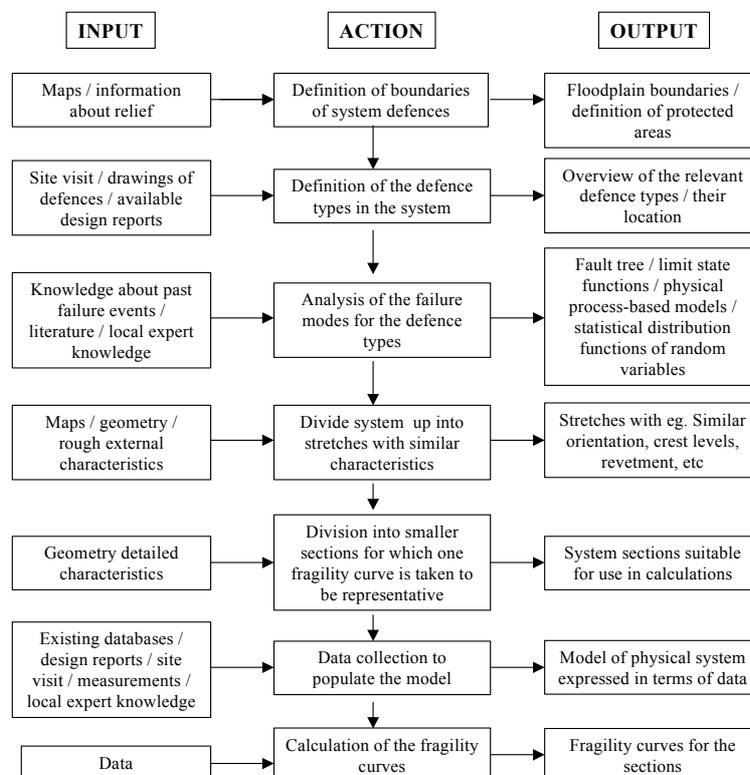


Figure 2 Steps in a system analysis of coastal defences to come to fragility, after Buijs et al, 2003

2.4. An introduction to the RASP tiered methodology and fragility curves

2.4.1 RASP

Interest in risk-based design and maintenance of flood defences has grown significantly in the UK in recent years. The main advantages of a risk-based safety approach are:

- It is based on the concept of risk and therefore considers all the aspects related to failure of a flood defence system: the strength and the 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.
- It supports the process of decision-making with respect to maintenance of a flood defence system as a risk-based analysis of flood defence systems points out the system's weak links and which parameters contribute most to the probability of failure (i.e. potential breach location). This knowledge enables the decision-maker to target maintenance activities.
- In case of large scale flood defence improvements the decision-maker can compare different design options in terms of the actual risk reduction and the costs which are associated with the improvement option.

To support a wide application of risk methodologies in flood defence an overarching risk-based framework (RASP) is being developed that integrates decisions on different levels (e.g. national, large-scale, strategy, scheme, etc) and across differing functions (local authorities, flood warning, operation and maintenance, etc.).

RASP stands for **R**isk **A**ssessment of flood and coastal defence systems for **S**trategic **P**lanning. It provides a tiered risk assessment methodology for systems of flood and coastal defences underpinning different levels of decision-making. RASP consists of a High Level Methodology informing national level decision making, an Intermediate Level informing regional decision making and a Detailed Level methodology informing decision making on the level of a flood defence system. Each level involves different types of decisions and has different data sources at its disposal. The tiered methodologies are being developed taking these different circumstances into account. An overview of the issues is given in the table below.

Table 1. Tiered risk assessment approach 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 type Condition grades Standard of Service Indicative flood plain maps Socio-economic data Land use mapping	Generic probabilities of defence failure based on condition assessment and crest freeboard Assumed dependency between defence sections Empirical methods to determine likely flood extent
Intermediate	<i>Above plus:</i> Flood defence strategy planning Regulation of development Prioritisation of maintenance Planning of flood warning	<i>Above plus:</i> Defence crest level and other dimensions where available Joint probability load distributions Flood plain topography Detailed socio-economic data	Probabilities of defence failure from reliability analysis Systems reliability analysis using joint loading conditions Modelling of limited number of inundation scenarios
Detailed	<i>Above plus:</i> Scheme appraisal and optimisation	<i>Above plus:</i> All parameters required describing defence strength Synthetic time series of loading conditions	Simulation based reliability analysis of system Simulation modelling of inundation

2.4.2 RASP and fragility curves

To develop an approach that considers the full potential risk distribution, rather than just limit state conditions, requires a relationship between defence performance and loading throughout the full range of loading conditions. This may be provided through the use of ‘fragility curves’. The following issues surrounding the concept of fragility are addressed:

- The definition of the concept of fragility
- The role of fragility curve in flood risk assessments
- Appearances of fragility curves in tiered risk assessments

Definitions

In RASP High Level Methodology^{vi}, the concept of fragility was first introduced as a tool in a system of tiered risk assessments.

The *fragility* of a structure was defined as the probability of failure conditional on a specific loading.

Role of the fragility curve in risk assessment in general

In a risk assessment the aim is to get an impression of the distribution of flood risk across a floodplain. Based on this distribution of risk, comparisons can be made where to target investments and which improvement options are most efficient in reducing the risk given an equal investment.

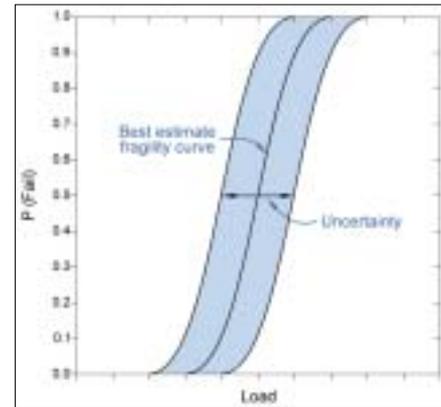


Figure 3: Example of a fragility curve

Risk is a function of the likelihood of an undesired event and the magnitude of the consequences given this undesired event. Purely considering flooding events, the likelihood is represented by the probability of failure of a flood defence given a certain set of loading conditions. The consequences are expressed by the damage caused by the flooding scenario that occurs under that set of loading conditions in the event of failure of the flood defence.

$$\{\text{Flood risk} \mid \text{flooding scenario}\} = P(\text{failure} \mid \text{loading conditions}) \times \{\text{damage} \mid \text{flooding scenario}\}$$

The fragility curve representing the probability of failure given a set of loading conditions is therefore a format that connects well with the approach to establish flood risk.

Appearances of fragility curves in a tiered risk assessment

The use of a fragility curve to represent flood defence performance may be applied at each of the three levels of RASP assessment. However, as the level of risk assessment connected to flooding gets more detailed, the analysis of the shape of the fragility curve and the flooding scenarios needs to be increasingly detailed and hence also the required information. Below an impression is given of how the difference in level of risk assessment works out for the fragility curves by contrasting the highest level and lowest level.

The highest level of risk assessment is relatively simple, easily applicable but still roughly representative of the distribution of flood risk. The main source of information (in the UK) is the NFCDD, a national database containing rough information about the flood defences. The fragility curves are taken to be generic for a limited number of types of flood and coastal defences. This background results in a fragility curve in which the loading conditions are expressed in one parameter on the horizontal axis.

In contrast, *the detailed level of risk assessment* is most complex, takes into account all failure modes irrespective of whether they are expected to contribute significantly or not and is based on comprehensive data. The fragility curves are determined for each flood defence section based on the local information. Instead of expressing loading in one parameter, the loading

conditions are approached in a more differentiated way. This results in conditional probabilities of failure given a certain water level, significant wave height, wave period, etc. This approach is graphically not very attractive but allows much more differentiation in flooding scenarios and accompanying damage levels.

Figure 4 below shows how the fragility curve sits within an overall risk based approach linking the ‘source’ of flooding, through the ‘pathway’ (in this case embankment) to the ‘receptor’.

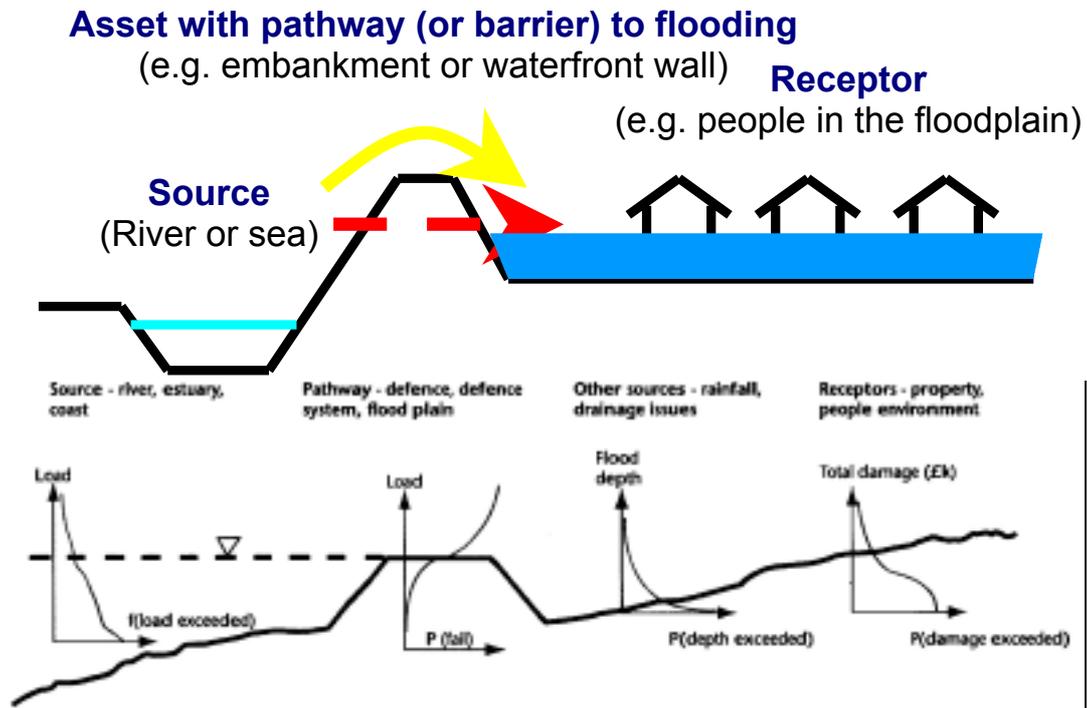


Figure 4 Source – Pathway – Receptor approach

3. Local factors affecting embankment performance

A logical assessment, based on cause/consequence relationships, will show that whatever the type of failure mode, the location of the failure is normally influenced by a combination of several causes or factors, although one of these may be dominant. The following sections consider different factors that may affect breach formation. Figure 5 below shows an example of how different features may affect embankment performance.

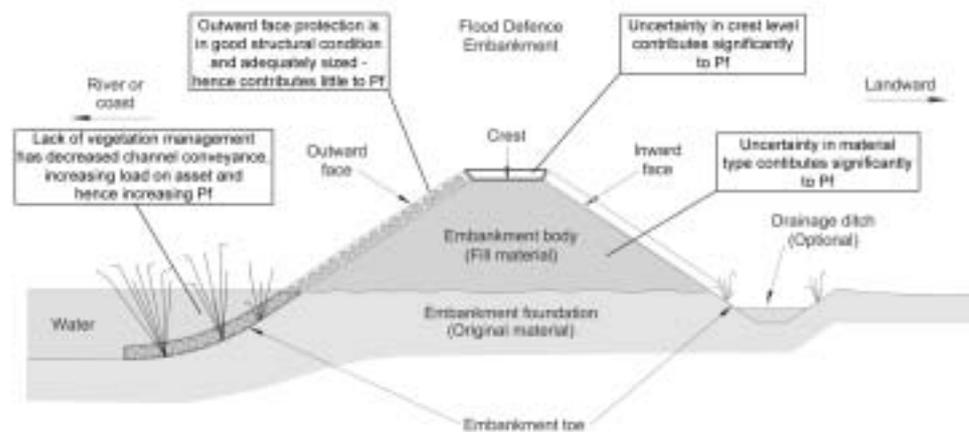
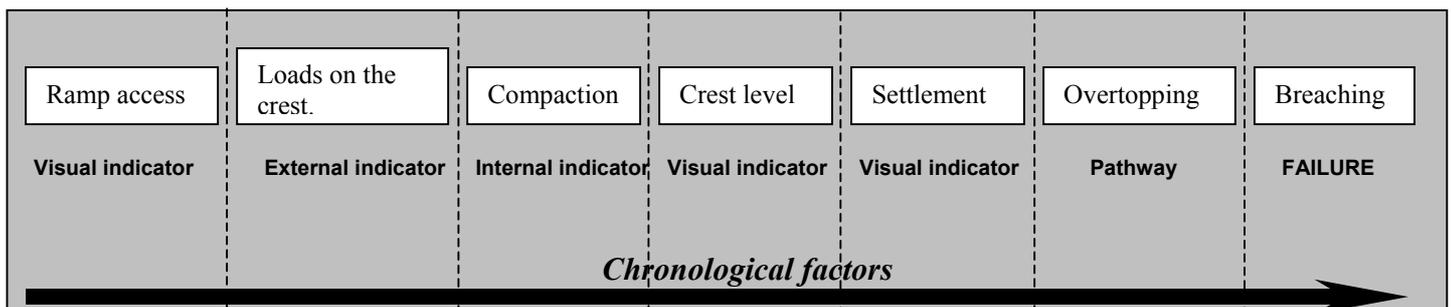


Figure 5 An example of prioritisation of components contributing most to defence failure, from Defra / Environment Agency (2004)

3.1. An interactive problem...

The following provides an example of factors leading to failure of an embankment. These have been ordered chronologically – another way of viewing interaction between factors on, say, a cause consequence diagram. The factors are also categorised according to possible type.



Although these indicators are sorted according to the time, the first one is not the most important one, since all play a vital role leading to embankment failure and hence all factors need to be considered in the future analysis.

The factors may be ranked within the following classes:

- **Internal factors:** these factors need a complete knowledge of the inner part of the embankment otherwise the embankment owner should extract a material sample in order to have an idea of the soil structure and composition.

- ***External factors:*** these factors could be detected by a visual survey; they are located on the surface of embankments.
- ***Environmental factors:*** these factors correspond to the physical elements surrounding embankments and affecting their overall performance.
- ***Human factors:*** these factors involve human actions and events; it can be as well physical actions toward embankments as lack of maintenance (legislation).

The following summaries have been drawn from reviews undertaken in Hungary as part of research into factors affecting breach formation under WP6. It was found that the data collected was insufficient (at this time) to allow a reliable statistical analysis of the role that the factors play. However, the work does provide an initial 'check list'.

3.1.2. Internal factors

Potential internal factors include:

	Factors affecting embankment performance	Example of risk generated by these factors
INTERNAL FACTORS	Moisture content	Lead to shallow instability due to variation of pore pressure
	Friction angle	A high friction angle will allow a higher resistance to failure and vice versa
	Grading	A bad gradation can make the embankment more permeable and leads to seepage and piping
	Dry unit weight	A material with a high dry unit weight could be less erodible than a low one and vice versa
	Plasticity index	A material with a poor plasticity index leads easily to piping failure
	Cohesion	Fine particle soils with low cohesion can be dislocated and washed away at lower speed of water than highly cohesive soils and then lead to a faster erosion of the body
	Permeable layers	The presence of highly permeable layer strata within embankment either leading to excessive seepage.
	Foundation interface properties	The foundation interface between two different materials is a sensitive area for piping, a weak interface material with a high lateral load due to unexpected rising of water level can lead to a translational sliding
	Compaction	Compaction is directly linked with permeability property of the embankment, in other word the piping phenomena is significantly governed by this factor
	High groundwater pressure	High groundwater pressure can trigger a deep rotational failure by acting in a permeable layer beneath the embankment
	Inner temperature	Temperature variations trigger cracks propagation by acting on natural dilatation and compression of the material

3.1.3. External factors

Potential external factors include:

EXTERNAL FACTORS	Reduction of crest level	The reduction of crest level increases considerably the risk of overtopping during flooding
	Settlement of the crest	Leads to a decrease of the crest level therefore to an increase of the overtopping risk
	Localised dipping of crest	Corresponds with a geometrical default of the crest which can be considered as a sensitive area for the breaching process
	Cracking, bulging, slumping of the surface protection	Corresponds with localized settlements on the surface protection, they lead to a global settlement of the embankment
	Sink holes on outward face	Visual factor which indicates an high risk of piping
	Vegetation properties and changes	The root mesh of this vegetation provides a network of reinforcement to the upper layer of the soil which improves resistance to erosion and to slope instability
	Longitudinal cracking along the crest	It is synonym of initiation of a deep rotational failure
	Bulging at the toe	The longitudinal cracking along the crest is generally combined with the bulging of the outward toe
	Steep slope and high crest level	A high embankment crest level with a very steep slope can't withstand efficiently the shear stress induced by the load of the water
	Raising of embankment	This raising will load the crest of the old embankment and could lead to loose the global stability of the structure then it can trigger a rotational failure
	Bare patches along embankment	This visual factor corresponds with a fragile area where erosion will have a higher development than in other embankment parts
	Channelling across crest and other outward face	These channels could be compared to small breach initiations therefore channelling process represents a high risk of breach formation during a flooding

3.1.4. Environmental factors

Potential environmental factors include:

ENVIRONMENTAL FACTORS	Gullies and headcuts	Corresponds with a previous overtopping, if an other overtopping occurs this part of the dam would be quickly eroded. Moreover, these factors will increase the global weakness of the rear face of the embankment
	Leakage with changes of water colour	A high risk of piping must be considered
	Animals (Rabbits, rats, moles etc.)	The burrow digs by these animals could quickly lead to a piping initiation
	Flow velocity	The sediment transport increases considerably with the flow velocity and the turbulences then the erosion peaks
	Water volume	it influences directly the mechanical equilibrium of the embankment then for example the lateral load of the water can trigger translational sliding failure
	Areas of standing and flooding water	These areas are exposed to constant seepage and high moisture content
	Waves properties	Wave height and velocity can as well erode as totally destroy embankments
	Flow content	A powerful flow can carry large and heavy objects as branches or rocks, these objects can greatly damage the structure integrity of embankments
	River morphology	River singularities generate turbulent sources which increase substantially the erosion rate
	Climatic conditions	Hailstones falls and heavy rain frequency affect embankment surfaces and lead to slope instabilities
	Overtopping	Acting on crest erosion and then trigger breach formation
	Trees growing throw embankments	The overall structure becomes non-homogeneous, every kind of sliding failure can occur within the embankment/tree interface, trees can also amplify wind effect
.....	

3.1.5. Human factors

Potential human factors include:

HUMAN FACTORS	Outfall structures	Outfall and culvert structures built through and into embankments induce weak points leading to failure if they were not well designed
	Embankment age	An old embankment has undergone a large amount of seasonal loads during his life which create natural cracks due to material dilatation and moisture evaporation
	Human crest loads	Cars and engines used by human on embankment cause longitudinal cracks and geometrical defaults and then rotational failure
	Ramp access	These ramps represent areas of frequent human loads
	Engine vibrations (Tractors, trains etc.)	Harmful factor for dry and stiff materials, formation of random cracks
	Wash from boats	Equivalent to waves
	Distance from human habitation	An overall factor could be estimated according to the distance from the nearest habitations. Habitations are in general harmful for embankments, they are sources of frequent people and vehicule movements, building operations etc.
	Agriculture	An embankment located next to agriculture fields can be affected by a constant maintenance of the field (fencing, watering) but also by farm animals living on. They can cause a severe embankment surface erosion due to a confined grazing during wet conditions
	Wood poles	Wood poles throw embankment create structure discontinuity due to their height, they include the transmission of dynamics effects of heavy wind onto embankment, leading to movement then to failure
	Legislation	A poor legislation can lead to a lack of maintenance, cracks and defaults expand without any survey

4. Methodology for identifying potential breach location

4.1 Concluding review

The problem of identifying breach location may be divided into two approaches:

- Assessing the relative risk of breach occurring based upon an assessment of the flood defence system
- Assessing the risk of breach formation at a specific location based upon local factors

4.1.1 Systems approach

A systems based approach has been developed in the Netherlands which uses limit state equations to define a number of possible embankment failure mechanisms. However, the current state of art concerning factors used in the limit state functions is quite limited. Indeed, the factors concerned are only factors which can easily and directly be measured with sensors or others measurement devices [e.g. water level, discharge, etc.]. Other factors exist that need to be integrated in these limit state functions [e.g. wind, volume, flow velocity, temperature variations etc.]. A detailed analysis of failure modes needs to be performed and afterward used to derive a complete set of limit state equations for the description of failure scenarios if the reliability of this approach is to be improved.

It has also been observed that certain factors are perceived to be very difficult to represent simply (e.g. animal burrowing, vegetation patches etc.). Nevertheless, this does not mean that they cannot be estimated through the use of a semi-quantitative approach.

The UK approach (RASP) adopts the concept of fragility curves to represent the performance of flood defences. The example in Figure 6 show possible curves for 5 different embankment condition grades (CG) and potential uncertainty around one condition grade curve. This uncertainty might be dependent upon a number of factors such as type of vegetation, embankment compaction, core material etc.

4.2 Conclusions

Since fragility curves build upon limit state equations, and we have seen that current knowledge regarding limit state equations is relatively low, we have a similar problem relating to the accuracy of fragility curves. In addition, the methods and assumptions underpinning the overall framework need to be recognised to avoid any misconceptions regarding overall accuracy. The accuracy of four components may be identified as key issues. These are:

- The quality of the process-based model and the way the model is represented in the limit state functions
- The representation of the uncertainties in the model and data
- The accuracy of the calculations
- The quality of the available data

4.1.1 Local approach

Efforts within IMPACT WP6 to correlate historic breach data (in Hungary) against different failure mechanisms / factors have not provided conclusive evidence of key and priority factors. However, correlation between location in a river catchment and typical size of breach has been established. It is anticipated that a more targeted analysis of a smaller set of breach events may still allow correlation of failure with various factors such as material type, condition etc.

Equally, an alternative approach by Partner 2 within WP2 of looking specifically at detailed embankment crest level against predicted flood water level, and relating embankment failure

to just depth of flow over the embankment, had a very successful outcome. This approach simply assumes that the dominant factor in the majority of failures is overflowing water, and hence analysis of this will predict the majority of failures.

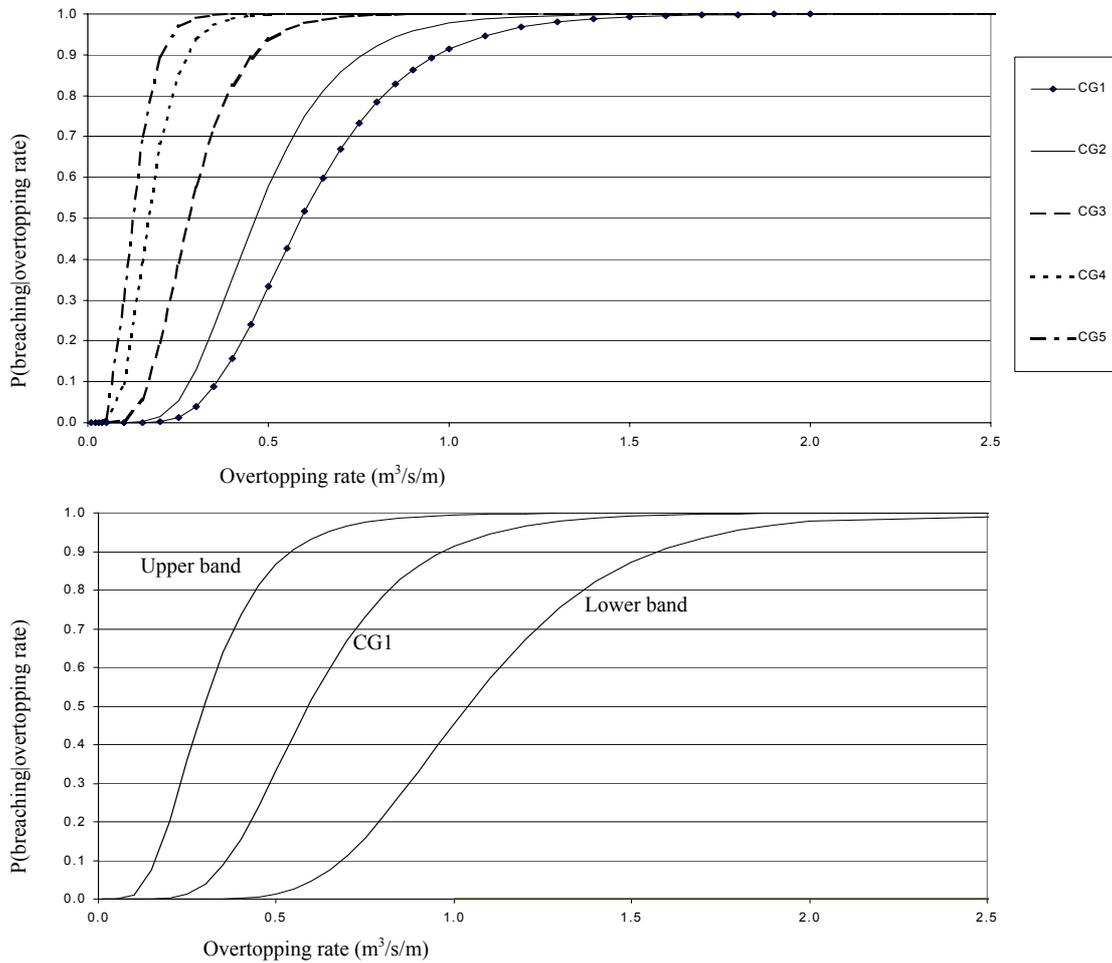


Figure 6 Example of fragility curves for broad scale risk assessments. Above fragility curves of 5 different condition grades (1=excellent, 5=very poor) for a narrow coastal impermeable embankment, turf front face and crest protection.

4.3 Conclusions

The following points may be concluded:

- 1 A framework for assessing the relative risk of breach occurring within a system of flood defences is provided within the RASP methodology being developed in the UK. This builds upon the earlier methodology of PC RING, currently in use in the Netherlands.
- 2 Whilst the RASP and PC RING approaches offer a framework for analysis, the reliability of the systems remains very dependent upon the limit state equations or fragility curves used and the assumptions underpinning the overall framework to represent embankment performance. These equations, curves and frameworks require validation and extension in order to refine overall model performance.

- 3 Data collected within IMPACT WP6 in Hungary may provide a useful base of information against which further refinement of fragility curves may be made. Analysis of the data to date has demonstrated a wide range of factors that affect embankment performance, leading to failure.
- 4 Focussing on one or two factors (e.g. overflowing water depth) may be sufficient to identify a majority of high risk locations, without the need to analyse large numbers of factors in detail. This approach requires further validation.

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Appendices

Appendix A PC-Ring

An introduction to PC-Ring

Technological developments of computers in the nineties have enabled the calculation of probabilities of failure of flood defence systems. The limit state functions applied in this software are based on former non-probabilistic reliability methods employed in the Netherlands.

The probabilistic calculations done with PC-Ring correspond with the detailed level methodology in RASP and take into account a large number of flood defence sections and multiple limit state functions. The main output from these calculations is:

- Probabilities of failure for each flood defence section included in the calculations and therefore an overview of the weak areas in the flood defence system.
- Insight in the key failure mode of a system of flood defences.
- Insight in the uncertainties associated with the parameters in the limit state functions and their contributions to the total probability of failure of the flood defence sections.

The local probabilities of failure can be used in determining the spatial distribution of risk. Defence improvements can be targeted to flood defence sections contributing most to the risk in the floodplain. The insight in which model parameter contributes most to the probability of failure of the flood defence section in combination with the limit state function provides advice on what kind of improvement activities are expected to be most effective. The effectiveness of different improvement options can be compared by analysing the reduction of risk in the floodplain associated with each option (see Buijs et al. (2003)^{vii}).

The reliability models applied in the limit state functions in PC-Ring are similar to those applied in the UK. The limit state functions of the failure modes implemented in PC-Ring are given as an example of how to fit current practice models for failure modes into limit state functions. Specific details can be found in Buijs (2003)^{viii} and translated from Vrouwenvelder et al. (2001) and in Lassing (2003)^{ix}. A very comprehensive discussion of limit state functions in connection to embankments can also be found in Kortenhaus & Oumeraci (2002)^x.

Introduction to the failure modes used in PC-Ring

PC-Ring only includes failure modes concerned with structural failure. The following failure modes of embankments are included in PC-Ring:

- Overtopping/overflow.
- Instability of the inside slope.
- Piping.
- Damage of the revetment on the outside slope and consequently erosion of the embankment body.

The fault tree of these failure modes related to an embankment is given in fig. 4.1.

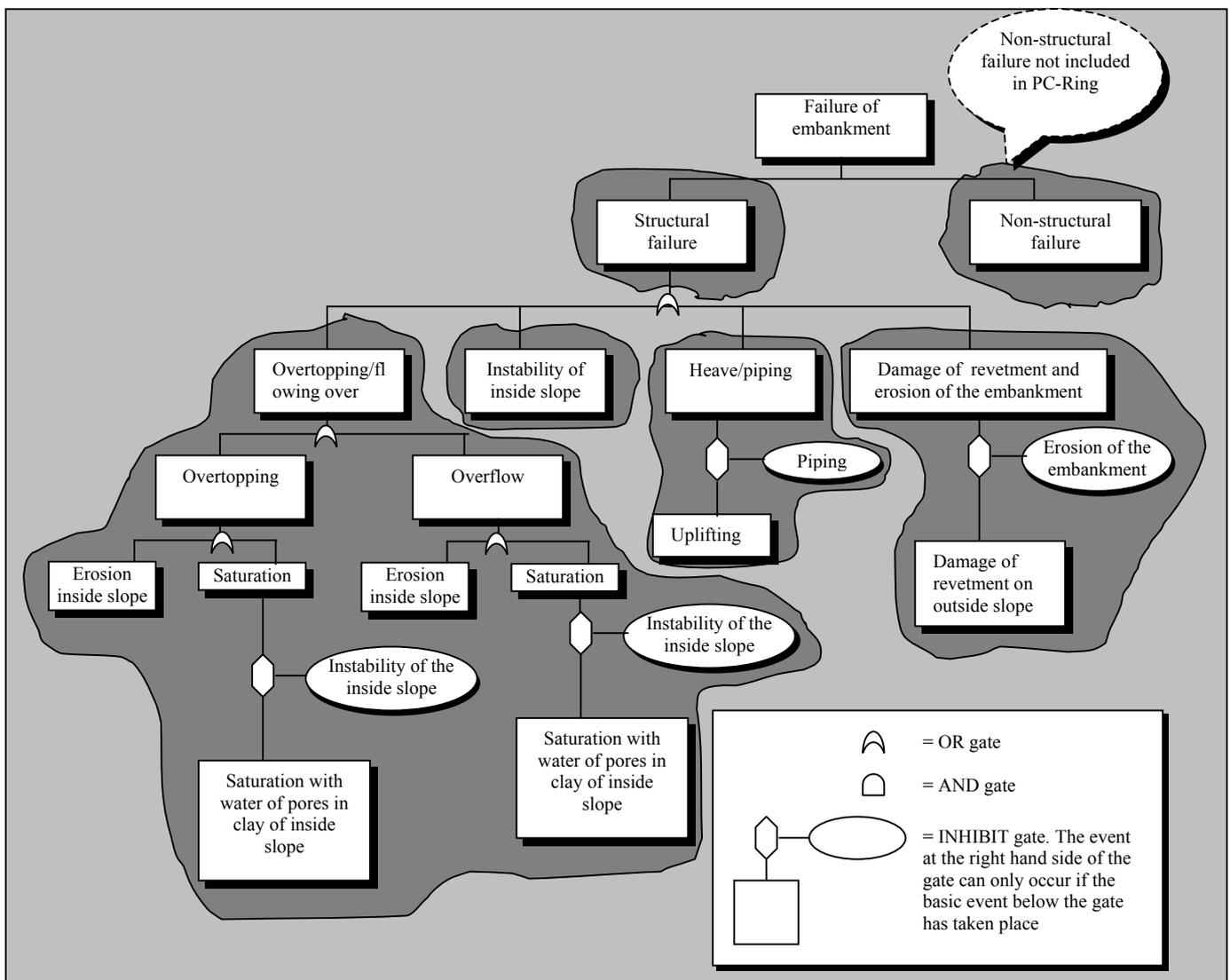


Figure 4.1. Failure modes of the embankment as applied in PC-Ring

Overtopping/overflow

Water discharges passing the crest of the embankment either due to overtopping or overflow are the cause of loading of the inside slope. Water discharges due to overflow are in PC-Ring only assumed to be relevant in case of off-shore wind or wave heights smaller than 1 mm. In the other situations the water discharges are assumed to occur due to wave overtopping^{xi}.

Failure of the inside slope due to the loading by the overtopping/overflow discharges can occur in two ways:

- Erosion of the inside slope.
 - Saturation of the pores in the clay and consequently instability of the inside slope.
- These failure modes are discussed below.

Erosion of inside slope

Water discharges due to overtopping or overflow respectively hit or scour the inside slope of the embankment. Due to this loading of the inside slope the grass gets damaged. After the grass has been damaged, the embankment body is exposed to the overtopping/overflow water. In the end, if this erosion process continues long enough, the embankment breaches. The duration of this erosion process depends on the duration of the storm.

- In case of *discharges due to overtopping* the limit state function 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 embankment, 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 percentage of time that overtopping occurs, this variable is applied to take the pulsating character of overtopping in account.

The critical discharge q_c can be determined in two ways: either a model based on the strength of the grass or the desired limit discharge can be entered manually.

The actual overtopping discharge q_o is calculated with the overtopping equations according to Van der Meer, revised version¹.

- In case of *discharges due to overflow* the limit state function is:

$$Z = h_d + \Delta h_c - h$$

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

The critical height Δh_c is derived from the critical overtopping discharge q_c , see Appendix 4-A. The latter is represented by the model based on the grass strength as is applied in case of failure due to overtopping.

The actual occurring water level h is available from the hydraulic boundary conditions.

Saturation

Water from the overtopping or overflow discharges infiltrates the pores of the clay soil of the crest and inside slope of the embankment. Increasing water pressures in the pores of the clay cause decreasing effective stresses and therefore, a decreasing shear strength. If the pores of the clay are completely saturated with water from the overtopping or overflow discharges the shear strength is at its lowest and the inside slope is most susceptible to instability. This description points out that failure consists of two events: the process of saturation of the pores in the clay and instability of the inside slope due to low shear strength.

The limit state function describing the process of saturation of the pores in the upper clay layer is:

$$Z_I = q_{cv} - m_{q_0} q_0$$

In which q_{cv} is the critical discharge for which within a few hours during a storm complete saturation of the upper layer of the inside slope occurs. The value of this q_{cv} depends on whether the water discharge is caused by overflow or wave overtopping. A water discharge due to overflow infiltrates the upper layer more progressively than due to wave overtopping. The extent of infiltration of the latter depends on the wave height. q_0 is the actual overtopping discharge according to Van der Meer revised. m_{q_0} is the model uncertainty with respect to the actual discharge.

The limit state function describing the process of instability of the inside slope due to the low shear strength caused by the high water pressures is:

$$Z_{II} = \tan(\alpha_c) - \tan(\alpha_i)$$

In which $\tan(\alpha_c)$ expresses the strength of the with water saturated clay layer at the inside slope, if this value is lower than the actual slope then instability of the inside slope occurs.

Instability of the inside slope

First, below a general description is given of how the stability of an embankment slope can be modelled. Second, the failure mode instability of the inside slope is presented.

Background information: stability according to Bishop

Instability of an embankment slope occurs if part of the embankment slides away along a slip plane. According to Bishop's approach this slip plane is circular and the stability is expressed in the form of a stability factor. To determine this stability factor the embankment is divided into vertical slices. The stability factor consists of a ratio between two moments that are taken with respect to the centre of this slip circle:

- The first moment is formed by the weight of the slices and the arm of

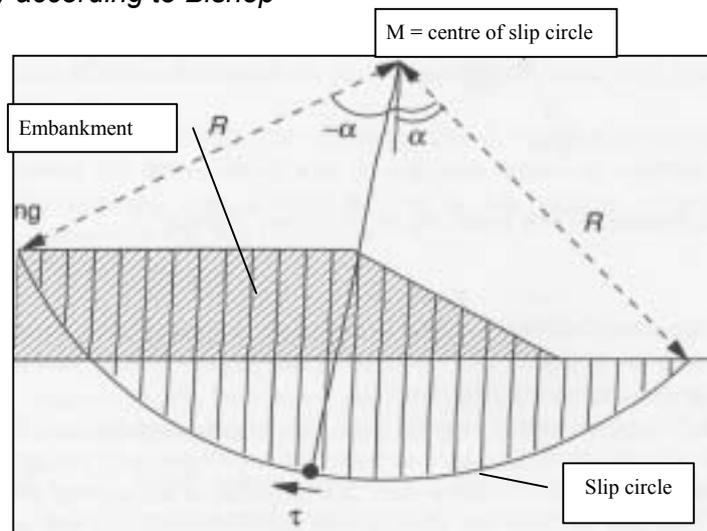


Figure 4.2. Slip circle model according to Bishop (From CUR162, 1999)

Uplifting and Piping

First uplifting causes openings in the impervious clay layer covering the sand layer. Second, a flow of water through these openings initialises an erosion process. This process can progress from the point of the openings caused by the previous uplifting behind the embankment towards the water outside. The erosion process takes the form of pipes undermining the foundation of the embankment. These pipes can eventually cause failure.

Uplifting

Uplifting occurs if the difference between the local water level h , and the water level “inside”, h_b is larger than the critical water level h_c , see fig. 4.4. This is expressed in the limit state function as:

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

In which m_o takes the model uncertainty of the model which determines h_c in account and m_h the level of damping. The critical water level expresses the limit water level for which almost uplifting occurs. This water level is based on the properties of the impervious layer

Piping

The embankment 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 .

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

In which 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.

Damage of revetment on the outside slope and erosion of the embankment body

The following types of revetments are discussed below:

- Grass.
- Placed stone revetment (directly on clay and placed on a granular filter).
- Asphalt revetments.

Grass

The waves load the outside slope of the embankment. If the grass gets damaged the body of the embankment is fully exposed to the loading by the waves and starts to erode. The body of the embankment consists of two parts: the clay cover layer and the embankment core. If both components are eroded, the embankment breaches. This failure mode is represented by the following limit state function:

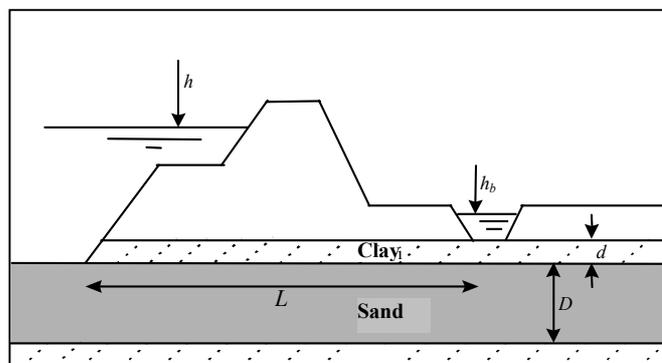


Figure 4.4. Parameters for uplifting/piping (from Vrouwenvelder et al., 2001)

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

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 embankment body. t_s is the duration of the storm.

Placed stone revetment

Two types of placed stone revetment are discussed below: placed directly on clay and placed on a granular filter. The chain of events is similar for both types of placed stone revetment. The waves load the revetment on the outside slope of the embankment. If the placed stones are damaged the body of the embankment is fully exposed to the loading by the waves and starts to erode. If the body of the embankment is eroded, breach occurs. The difference between stone revetment placed directly on clay and on granular filter is the description of damage of the revetment in terms of limit state functions.

Stone revetment placed directly on clay

The failure mode damage of the stone revetment placed directly on clay is represented by the limit state function:

$$Z = c_k \Delta D - r H_s$$

In which c_k is a coefficient for the strength of the placed stones, Δ the relative density and D the thickness of the placed stones. Apart from this, the reduction factor r and the significant wave height H_s are part of the limit state function.

Stone revetment placed on granular filter

Failure of the stone revetment placed on granular filter is described by two limit state functions, represented by $Z_{b1} < 0$ OR $Z_{b2} < 0$.

$$Z_{b1} = \frac{c_f \Delta D^{1.67} \Gamma^{1.67}}{(\Lambda \tan \alpha_u)^{0.67}} - \frac{r H_s}{(r S_{op})^{0.33}}$$

In which c_f is a coefficient for the strength of the stone revetment, Δ is the relative density, D is the thickness of the revetment, Γ is a factor of influence of the friction between stones, inertia, etc., Λ is the leakage length, α_u is the angle of the slope, S_{op} is the wave steepness, H_s is the significant wave height and r is a reduction factor taking the obliqueness of the waves into account.

$$Z_{b2} = c_{gf} \left(\frac{\tan \alpha_u}{\sqrt{S_{op}}} \right)^{-2/3} - \frac{H_s}{\Delta D}$$

In which c_{gf} is a coefficient for the strength of the revetment and the other parameters are discussed with the first limit state function.

Erosion of the embankment body leading to breach

Erosion of the embankment is represented by the following limit state function:

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

In which 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 embankment body. t_s is the duration of the storm. The models that are used to determine these values are equal to those applied in the failure mode of revetment type grass.

Asphalt revetments

Asphalt revetments fail either due to uplifting by water overpressures or due to wave impacts, see fig. 4.5. After failure of the revetment the embankment body is fully exposed to loading by the waves. The embankment breaches after the body of the embankment has completely eroded.

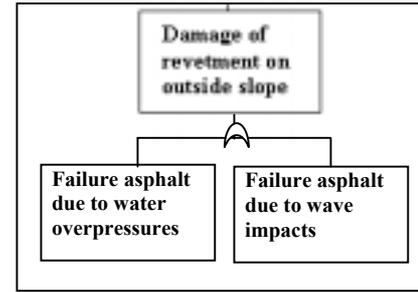


Figure 4.5. Failure mode damage asphalt and erosion core of embankment in PC-Ring

There are three main types of asphalt revetment: open stone asphalt, fully penetrated and saturated riprap revetment, partially penetrated riprap revetment. For these types the limit state function of failure due to uplifting is equal but that of failure due to wave impacts differs.

Failure of asphalt revetment due to uplifting

Failure of asphalt revetment occurs when the pressure difference over the revetment at the level of the water line exceeds the weight of the asphalt. This leads to the following limit state function:

$$Z = \Delta D - 0,21 Q_n (a + v) R_w \quad (\text{from Technisch rapport Asphalt voor Waterkeren (2000)}^{\text{xiii}})$$

In which Δ is the relative density, D is the thickness of the asphalt, Q_n is a factor depending on the angle of the outside slope α_u , a is the vertically measured distance between the toe of the impermeable revetment and the outside water level, v the vertically measured distance between the outside water level and the groundwater level and R_w a reduction factor in connection to the position of the outside water level. For background information, see also chapter 9 in Pilarczyk (1998)^{xiv}.

Failure due to impact of waves: open stone asphalt

Failure of open asphalt revetment due to the impact of waves can be represented by the following limit state function:

$$Z = D - D_{\text{required}}$$

In which D is the thickness of the open stone asphalt and D_{required} can be determined according to a diagram which plots the required asphalt thickness as a function of the significant wave height for different slopes and clay or sand soil (see TAW (1985)^{xv}).

Failure due to impact of waves: fully penetrated and saturated riprap revetment

Failure due to wave impact of fully penetrated and saturated riprap revetment is considered to be an irrelevant failure mode.

Failure due to impact of waves: partially penetrated riprap revetment

The limit state function for failure of partially penetrated riprap revetment due to impact of waves is given by:

$$Z = D_{n50} - \frac{H_s \xi_p^b}{\Delta_m \psi_u \Phi_{sw} \cos(\alpha)}$$

In which D_{n50} is the nominal diameter for which 50% of the weight of the grains is larger or smaller than this value, ξ_p is the breaker parameter, Δ_m is the relative density, ψ_u is a parameter for penetration of the asphalt, Φ_{sw} is the stability factor and α is the angle of the outside slope.

Erosion of the embankment body leading to breach

The limit state function of the erosion of the embankment body after being fully exposed to the wave loading is:

$$Z = t_{RB} - t_s$$

In which t_{RB} is the time needed to erode the core of the embankment and t_s is the duration of the storm.

Structures

In PC-Ring the failure modes considered for structures are:

- Piping underneath structures
- Failure of closure of the water retaining gate (moving structure)

Piping underneath structures

The limit state function applied for piping underneath structures is based on the classical approach according to Lane (more sophisticated methods are under development):

$$Z = m_L L - c_L m_c (h - h_b)$$

$$L = L_v + L_h/3$$

In which c_L is the coefficient according to Lane, h is the outside water level, h_b is the inside water level and m_L and m_c are parameters that take the model uncertainty into account. L_v and L_h are respectively the vertical and horizontal seepage lengths. For structures founded on piles $L_h = 0$.

Failure of closure of the water retaining gate

The fault tree for this failure mode is given in figure 4.6. The main thought behind this fault tree is that if the gate fails to close, problems start occurring when the volume of water flowing inland exceeds the available volume of storage.

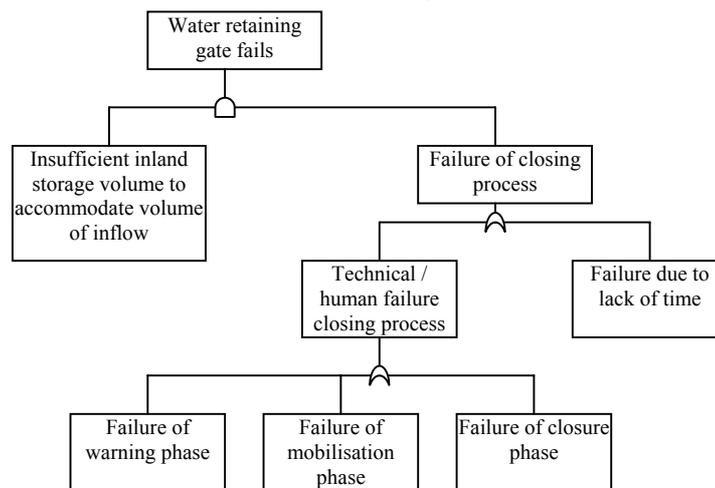


Figure 4.6 Fault tree of failure of closure of a water retaining gate

Insufficient inland storage volume

The limit state function representing insufficient inland storage volume to accommodate the volume of water flowing in due to failure of the water retaining gate is as follows:

$$Z = m_{stor} A_{stor} h_{pv} - m_{in} t_s Q_{in}$$

In which $m_{stor} A_{stor} h_{pv}$ represents the inland storage volume, consisting of A_{stor} which is the area of the available storage, h_{pv} which is the allowed increase in water level in the storage area and m_{stor} is a factor taking the model uncertainty into account. $m_{in} t_s Q_{in}$ represents the volume of water flowing in because of the failing water retaining gate, consisting of Q_{in} which is the discharge of the water flowing in, t_s which is the duration of the storm and m_{in} which is a factor taking the model uncertainty into account.

Failure due to lack of time

Failure due to lack of time occurs when the time required to close the gate exceeds the maximum available time. The limit state function is:

$$Z = MAT - T_{cp}$$

In which MAT is the Maximum Available Time and T_{cp} is the time required to close the gate. T_{cp} is the sum of the required time for the warning, mobilisation and closure phase. The maximum available time is determined partly by the time between the warning for the storm and the actual start of the storm. The other part of the maximum available time is formed by the time required to fill the available inland storage volume (see previous limit state function).

Technical / human failure

Probabilities of failure due to technical / human failure can be established according to for instance Cooke et al. (1998)^{xvi} and Van Elst (1997)^{xvii}. The probability of technical / human failure is a combination between failure in the warning, mobilisation and closure phase, see below. The accompanying reliability index can be determined according to the second equation.

$$F_{nc} = F_W \cup F_M \cup F_{CF}$$

$$\beta_{nc} = -\Phi^{-1}(P(F_{ns}))$$

The limit state function for technical / human failure is given by:

$$Z = \beta_{nc} - v_{nc}$$

For β_{nc} see equations mentioned above and v_{nc} is a variable with mean 0 and standard deviation 1 which represents the intrinsic uncertainty.

2.3.2.1. Dune

During a storm the front face of the dune erodes and the eroded material is deposited on the foreshore of the dune. Breach occurs when the remaining profile is insufficient to withstand storm conditions. The profile of the dune as a function of the loading conditions during storm is predicted with the model according to Vellinga (1986)^{xviii}, see figure 4.7.

In PC-Ring failure of dunes is approached by comparing the initial profile of the dune and a minimum allowed profile as a result of the storm, see figure 4.7. The minimum allowed profile under a storm is represented by the line BCDEF, for which the stretch between D and E is determined according to Vellinga (1986), the formula is given in the figure. The limit state function establishes whether the initial profile is sufficient to provide the minimum allowed profile:

$$Z = m_D V_1 + V_2 - V_3$$

In which V_1 , V_2 and V_3 are defined according to figure 4.7 and m_D is a factor taking the model uncertainty into account.

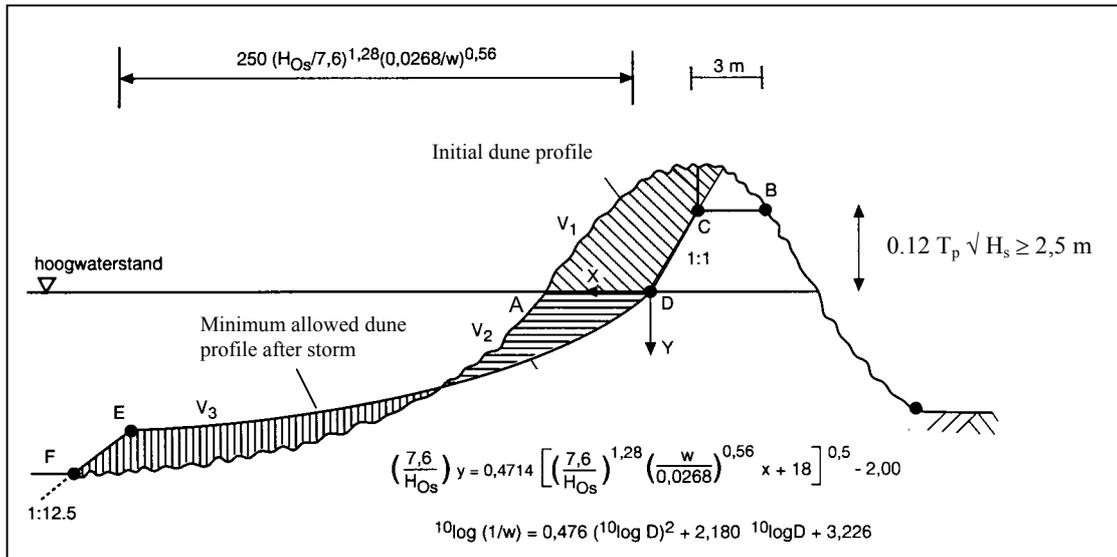


Figure 4.7 Comparison of the initial profile of the dune and the minimum allowed profile as a result of the storm (from Vrouwenvelder et al. (2001))

Statistical models

The statistical models that are applied for the random variables in PC-Ring are described below. First the statistical models of random variables in PC-Ring in general are discussed. Second the statistical models of the hydraulic boundary conditions are presented separately.

Statistical models of random variables in PC-Ring in general

The statistical models consist of the following components:

- Statistical distribution functions.
- Spatial correlation function.
- Model representing the correlation in time.

These components are discussed below.

Statistical distribution functions

For each variable in the above mentioned limit state functions a choice has to be made whether the variable is of a deterministic or random nature. In the latter case the distribution functions have already been established in the computer code and cannot be altered by means of input. However, the parameters accompanying the distribution function in the form of mean values and standard deviations have to be defined. Instead of the standard deviation regularly the expression of the variation coefficient, $V = \sigma/\mu$, is used. Apart from the random variables related to the hydraulic boundary conditions, most random variables in PC-Ring are normal or lognormal distributed.

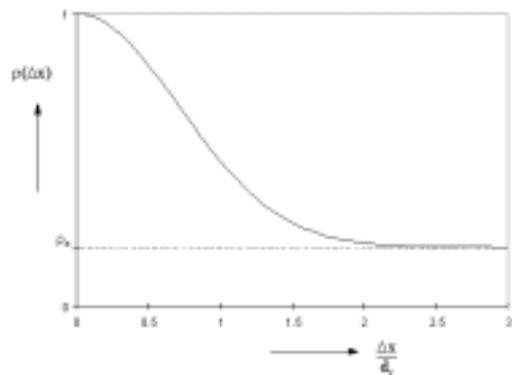


Figure 4.8. Shape of the correlation function (from Vrouwenvelder et al., 2001)

Spatial correlation function

The in PC-Ring applied spatial correlation function is^{xix}:

$$\rho(\Delta x) = \rho_x + (1 - \rho_x) \exp\left\{-\frac{\Delta x^2}{d_x^2}\right\}$$

In which ρ_x is a constant correlation and d_x is the correlation distance. In fig. 4.7. an impression is given of the shape of the function.

Model representing correlation in time

In PC-Ring the Borges Castanheta model is applied to represent the correlation in time. According to this model the time is divided into intervals Δ_t in which complete correlation is assumed. Between the time intervals a constant correlation d_t is taken. An impression of this model is given in fig.4.9.

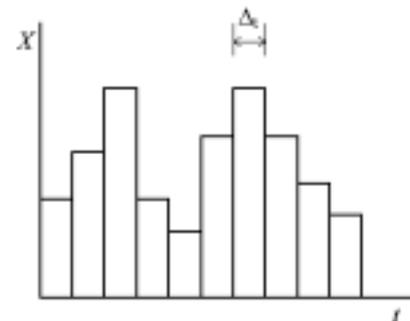


Figure 4.9. Representation of the Borges Castanheta model (from Vrouwenvelder et al., 2001)

Statistical models of hydraulic boundary conditions

In PC-Ring a number of models for different situations of hydraulic boundary conditions are incorporated. As an example below the model for a tidal river is presented. In fig. 4.10. and fig. 4.10. the model that is applied to determine the local hydraulic boundary conditions and

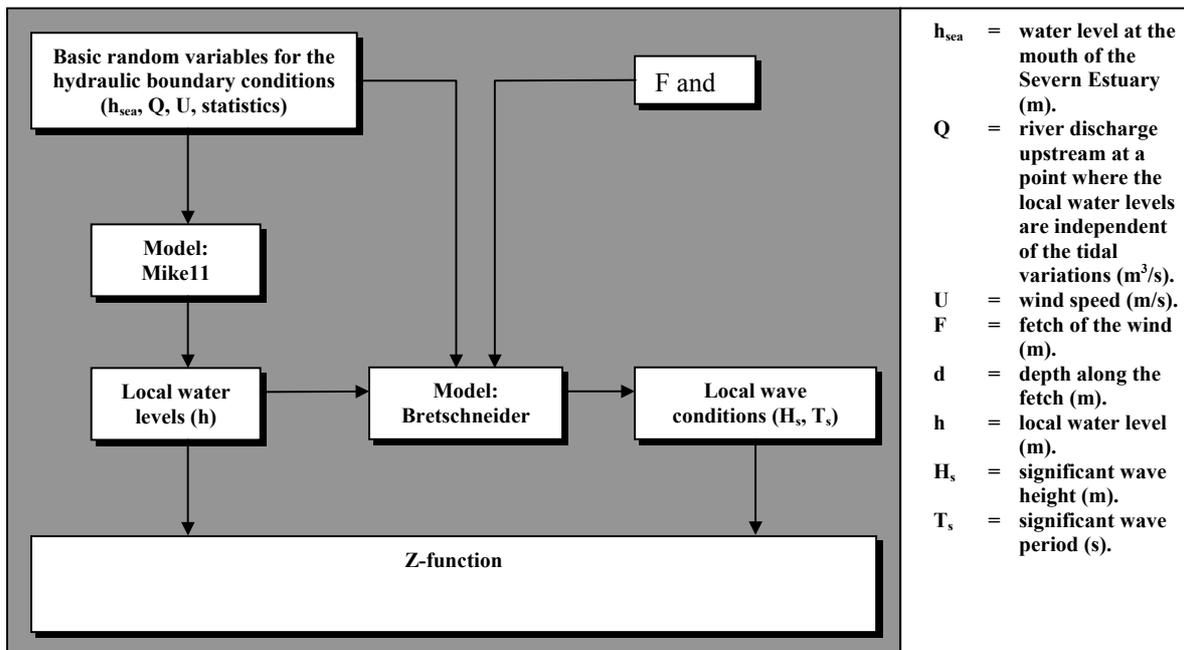


Figure 4.10. Model that is applied to determine the local hydraulic boundary conditions in the Z-functions

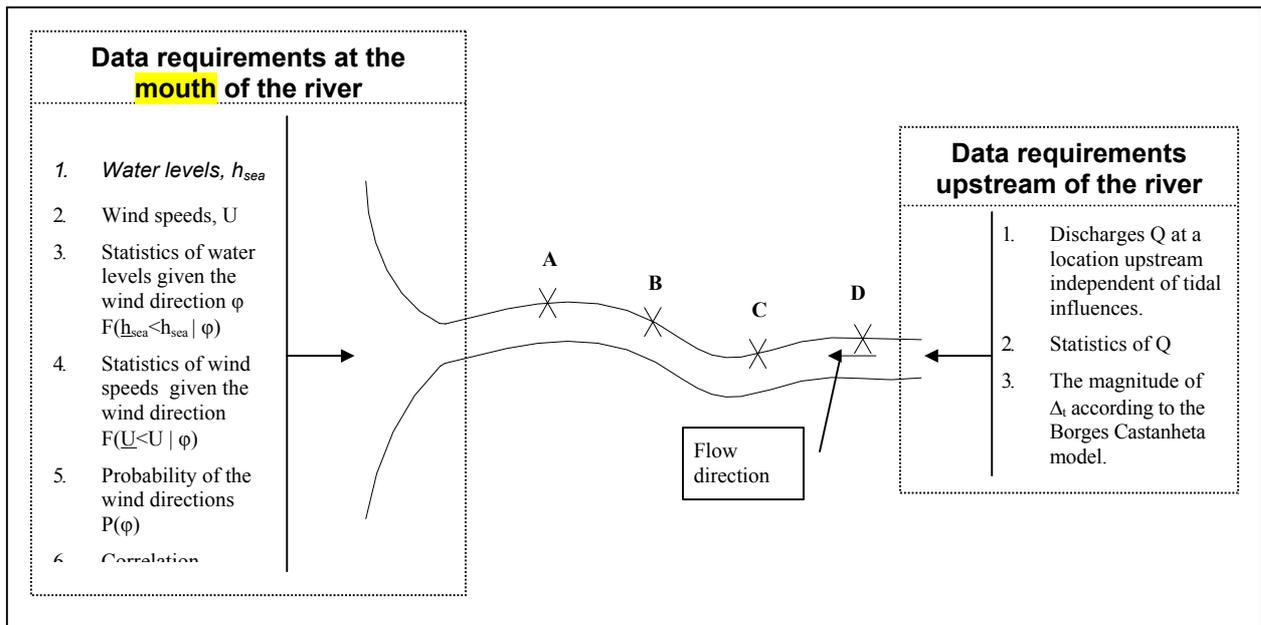


Figure 4.11. The data requirements of the basic random variables and the statistics that are used to determine the local hydraulic boundary conditions at for instance locations A, B, C and D

the data requirements for this model are presented. Below the following aspects of this model are further explained:

- The way in which the local water levels are determined in PC-Ring.
- The statistics that are mentioned in fig. 4.11.

Local water levels

To explain the model for the tidal river as applied in PC-Ring fig. 4.11. is used as an example. The basic random variables h_{sea} , Q and U can occur in various different combinations. Each different combination results in a different local water level. The local water levels at for instance locations A, B, C and D are predicted as a function of different combinations of the basic random variables using Mike11, sobek, etc.. The results of these calculations are laid down in one table for each location A, B, C and D. To determine the

local water level for flood defences situated for example between A and B, PC-Ring linearly interpolates between the water level at A and the water level at B. This linear interpolation takes place according to the distance of the location of the flood defence with respect to A and with respect to B.

The combinations of the basic random variables are formed by:

- *Nine different discharges.* The discharge values are chosen such that the range of occurring discharges is sufficiently represented. The extreme values are emphasised in this choice.
- *Five different wind speeds.* The wind speed values are chosen such that the range of occurring wind speeds is sufficiently represented.
- *A number of wind directions:* those wind directions are chosen that are relevant to the probability of flooding.
- *Six different water levels at sea.* The water level values are chosen such that the range of occurring water levels is sufficiently represented. The extreme values are emphasised in this choice.

Statistics

The following statistics are applied in PC-Ring with respect to the hydraulic boundary conditions:

- *Water levels at sea given the wind direction:*

For $h_{sea} > m_d$:

$$F_{weibull}(h_{sea} | \varphi) = P(h_{sea} < h_{sea} | \varphi) = 1 - p_c \exp\left(-\left(\frac{h_{sea}}{\sigma}\right)^\alpha + \left(\frac{m_d}{\sigma}\right)^\alpha\right)$$

In which h_{sea} is the water level at the sea, φ is the wind direction and p_c , m_d , σ , α are variables that determine the shape of the distribution function.

- *Wind speeds given the water level and the wind direction:*

$$F(u | h_{sea}, \varphi) = P(u < u | h_{sea}, \varphi) = \exp\left[-\exp\left(\frac{-K_\varphi(u) + \rho_w * (h_{sea} - A_h) / B_h}{m_w}\right)\right]$$

$$K_\varphi(u) = a_w u^2 + b_w u + c_w$$

In which u is the wind speed, h_{sea} is the water level, φ is the wind direction, ρ_w is the correlation between the wind speed and the water level given a wind direction, A_h , B_h and m_w are fitting parameters. This model for the correlation between the wind speed and water level originates from the model developed by Volker.

- *The probability of the wind directions:*
The wind direction is a discrete random variable. The statistics consist of a probability of each of the 16 wind directions.
- *The discharge Q:*
the statistics of the discharge Q are represented by Q as a function of the return period. This function is in the form of:

$$Q = a * \ln(R) + b$$

In which Q is the discharge and R is the return period of Q . a and b are fitting parameters. The statistics of Q can consist of more than one connecting functions of the same form as presented above. The statistics of Q cannot be entered as variables in PC-Ring but are fixed in the computer code.

Available calculation methods in PC-Ring

First an overview is given of the main steps that PC-Ring takes to come to an annual probability of flooding of a flood defence system. Second the main calculation methods are mentioned that are applied. The information below is taken from Vrouwenvelder (1999)^{xx}.

Main steps taken in PC-Ring calculations

The following steps are taken in the calculation of the flood defence system's probability of flooding:

9. Calculation of the probability of failure of one flood defence cross section for one tide, one partial failure mode (for instance failure mode overtopping, partial failure mode saturation), given the wind direction.
10. Combination of the partial failure modes resulting in the probability of failure of one total failure mode.
11. Taking the probability of the wind directions into account.
12. Determining the probability of failure due to one failure mode for the total flood defence stretch for which the under step 1 mentioned flood defence cross section is representative.
13. Combining the probabilities of failure of all the wind directions.
14. Determining the probability of failure for the total regarded period.
15. Combining the probabilities of the different failure modes.
16. Combining all the flood defence stretches to find a total flood defence system's probability of flooding.

Calculation methods as applied in PC-Ring

When regarding the above mentioned eight steps to calculate the system's probability of flooding, two main calculation methods occur:

- The calculation of the probability of failure represented by one limit state function, as in the above mentioned step 1.
- The combination of different limit state functions taking mutual correlations into account, as in the above mentioned steps 2 to 8.

Probability of failure of one limit state function

In PC-Ring the below mentioned main methods are available to calculate the probability of failure of one limit state function:

- FORM (First Order Reliability Method)
- SORM (Second Order Reliability Method)
- MC (Crude Monte Carlo)
- DS (Directional Sampling)
- A number of combinations of the above mentioned methods: for instance an option that PC-Ring automatically switches to DS if convergence does not occur in a calculation with FORM or SORM. Other options involve the combination of DS and FORM, the latter method is then used to find the design point.

Combination calculations

Consider a system consisting of n elements. An element can for instance be: a cross section, a tide, a wind direction, a stretch, a failure mode. Each element is represented by one Z -function. Two elements of the system are picked and are combined to form one equivalent representative element. In other words two Z -functions are combined to one. The total amount of elements in the system is reduced from n to $n-1$. Repeating this procedure over and over again will eventually reduce the amount of elements in the system to one. In other words, the system is "wrapped up".

The procedure to find one equivalent representative Z -function is according to the method of Hohenbichler^{xxi}. This procedure calculates $P(Z_1 < 0 \text{ AND } Z_2 < 0)$ taking the

mutual correlation into account. If this probability is known, then $P(Z_1 < 0 \text{ OR } Z_2 < 0)$ can be determined.

Limiting the number of flood defence sections in calculations

The process of flood defence modelling and data gathering presents time-consuming activities in practice. The main thought behind solving this practical problem is considering the flood defence system as a serial system. This main thought results in a practical approach to select the appropriate cross sections for the calculations with PC-Ring.

Practical problem: laborious flood defence modelling and data gathering

In order to make the calculations with PC-Ring, the flood defence system must be translated into a model. This model can be expressed in data, and these data are used in the calculations. Ideally, the complete flood defence system is thus expressed in data and taken into account in the probability calculations. However, in practice this takes a lot of time in terms of flood defence modelling and data gathering. This time is usually not available.

Serial system and weakest link

A way to deal with this problem is to regard the flood defence system as a serial system. The weakest link in a serial system dominates the total system's probability of failure. Therefore, only the cross sections are taken into account that are expected to contribute most to the total system's probability of failure.

Practical approach of flood defence modelling and data gathering

The process of cross section selection and data gathering is as follows:

1. Division of the water defence system in *defence types*.
2. The next step is to divide the water defence system in *embankment stretches*. To this end rough information and insights are used. The division is based on the external physical characteristics and not yet on the characteristics directly connected to the failure modes, although implicitly the connections are there. The following characteristics are important for this selection^{xxii}:
 - Orientation to the wind directions: embankment stretches that are orientated differently will be loaded by different wave regimes and therefore must be discerned as different embankment stretches.
 - High water regimes, differences in extreme water levels: lengths of the water defence system for which different high water regimes are relevant must be discerned as separate embankment stretches.
 - Geometrical characteristics foreshore: Embankment stretches with significant different sizes of the foreshore in terms of height and width.
 - External geometry of the water defence: lengths of the water defence system with significant differences in height and (external) construction are discerned as different embankment stretches. Differences in geometry will lead to differences in loading conditions due to the same hydraulic boundary conditions.
3. The water defence system has now been divided into rough stretches with the same type of characteristics. However, cross sections have to be determined which can be regarded as representative of the total embankment stretch. If still significant differences between cross sections in an embankment stretch are present, then the stretches need to be divided into further parts: *the flood defence sections*. A first insight with regard to weak spots in the water defence system can be given by managing authorities. These authorities can also help to determine the relevant failure modes for certain embankment sections. The following failure mode related considerations can form a basis for a division in embankment sections:
 - Different types of outside slope revetment (types and construction), this will lead to a further division for the failure mode: damage of the revetment on the outside slope and erosion of the embankment body.

- Differences in geometry on a detailed level has consequences for each of the failure modes.
 - Differences in the foundation soil can have a considerable effect on the contributions to the probability of failure of geotechnical failure modes. For these failure modes a further division has to be made.
 - Differences in the inside slope revetment of embankments (quality of the grass, thickness and qualification of the clay cover layer on the inside slope, the angle of the inside slope, etc.), this kind of information is especially useful for the division in embankment sections for the failure modes overtopping and consequently erosion of the inside slope or instability of the inside slope.
 - Information about the construction of the embankment (clay embankment, sand embankment with a clay core, etc...) and information about the soil layers underneath and directly next to the embankment (in front of and behind the embankment), this information is relevant to the failure modes heave and piping, instability of the inside slope and damage to the revetment and erosion of the embankment body.
4. After step 3 the water defence system has been divided in embankment sections. The following step is to *select the relevant sections* for the reliability analysis. The procedure to come to this selection of sections is given below:
- Regard a failure mode for which it is desirable to reduce the number of embankment sections. The first logical step is to eliminate all the embankment sections for which the mode is not relevant, or in other words: the contribution of the section to the probability of failure due to a certain mode is negligible in advance. Well known weak spots can provide valuable first insight in which sections contribute significantly and which ones do not.
 - For the remaining sections, indicators are used to rank them. These indicators are related to the failure modes. The number of selected sections can be limited based on this ranking.
 - Apart from the selected weak sections based on the indicators, sections from the middle and strong categories have to be chosen. This is done because it is practical to be able to make an estimate of the probability of flooding after eliminating the weak spots in the probability of failure calculations.
 - The former three steps have to be performed for all failure modes, which can result in a different selection of embankment sections for each mode.
 - Check the spreading of the sections along the water defence system with respect to the magnitude of the expected consequences. The sections with substantial consequences that fall out off the analysis should be added or shift a bit with the choice in the middle and strong sections. With the total risk analysis in mind, a strong section with extensive consequences can contribute just as much or even more to the total risk as a weak section with hardly any consequences.
5. The first selection of embankment sections has been finished. For each section it is clear which failure modes are regarded. The next step is to *gather data* for the regarded failure modes and embankment sections.
6. After the first calculations of the probability of failure with PC-Ring a check is made if sections have to be added or adjusted:
- First the contribution of the failure modes to the total probability of failure is regarded.
 - For the mode with the largest contribution a check has to be made if the last selected sections still have a significant contribution to the probability of failure. If they do an additional selection of sections for that mechanism has to be made. This check needs to be made for all the failure modes with an emphasis on the ones with the largest contribution.

This process has to be repeated until no more sections need to be included in the analysis. The number of cycles depends on the number of embankment sections which are chosen initially and after expansion.

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