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Reliability analysis of flood defence structures and systems in Europe

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ABSTRACT: In this paper the reliability analysis of flood defence systems and the probabilistic flood risk analysis approach are outlined. The application of probabilistic design methods offers the designer a way to unify the design of engineering structures, processes and management systems. For this reason there is a growing interest in the use of these methods in the design and safety analysis of flood defences and a separate task on this issue in the European **FLOOD*site*** project was defined. This paper describes the background of probabilistic analyses, uncertainties , and system analysis and how this has been dealt with under **FLOOD*site***. Eventually, a case study at the German Bight has been used to illustrate the application of the tools and results are discussed here as well.

1 INTRODUCTION

Flood defence systems of flood-prone areas can be represented by fault trees. An example is given in Fig. 1. Failure of the subsystems (dike, dune sluice, levee) of the system leads to flooding of the polder area. The subsystems all consist of elements (e.g. sea dikes or river embankments can be divided in different sections). Failure of any of these elements of the subsystem "dike 1" leads to flooding of the hinterland. For all elements of the flood defence system all failure modes for such elements can be the cause of failure. The most important failure modes have been addressed and modelled in Task 4 of **FLOOD*site*** (Morris et al. 2008 and Allsop et al. 2007). A reliability tool has been developed in **FLOOD*site***, which is able to calculate the failure probability of the top event (inundation, see Fig. 1) with a Monte Carlo simulation approach, for any construction type, configuration of the fault tree, and for a large number of probability density functions of load and resistance variables. This paper shows the applicability of the reliability tool to a case study at the German Bight in Germany. The paper starts with an introduction to probabilistic analysis and uncertainties. It continues by addressing the tools for a probabilistic systems analysis and introduces the calculation methods used under **FLOOD*site***. The paper ends with a case study of a flood defence system and a list of recommendations for further improvements and research. Readers

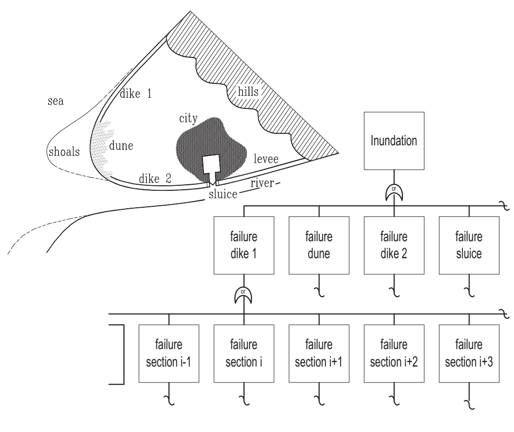

Figure 1. Flood defence system and its elements presented in a fault tree.

are referred to the **FLOOD*site*** documents T07-08-01 and T07-08-02 for a complete set of results of Task 7. These documents are available from the **FLOOD*site*** homepage under www.floodsite.net.

2 PROBABILISTIC ANALYSIS

Typically two types of failure are of interest in a flood defence reliability analysis: the annual probability of failure and the event probability of failure, conditional on the applied load (fragility). The former measure provides information that can be used directly to determine the state of the defence, the latter are derived for use in flood system risk analysis models (see Gouldby et al. (2008) for example).

These types of failure can be determined by analysing historical failure data and by probabilistic calculation of the limit states. For most cases there is not enough specific failure data available so we have to determine the failure probabilities by computation. A limit state function is a function of the strength and the load for a particular failure mode. In general the formulation of the limit state function is: Z = R−S in which R is the strength and S is the load. The failure mode will not occur as long as the limit state function is positive. The line Z = 0 is a limit state. This line represents all the combinations of values of the strength and the loading for which the failure mode will just not occur. So it is a boundary between functioning and failure. In the limit state function the strength and load variables are assumed to be stochastic variables. A stochastic variable is a variable which is defined by a probability distribution and a probability density function. The probability distribution F(x) returns the probability that the variable is less than x. The probability density function is the first derivative of the probability distribution.

If the distribution and the density of all the strength and load variables are known it is possible to estimate the probability that the load has a value x and that the strength has a value less than x. Typically, the loading events are representative of a specific duration and the total number of potential events per year is obtained, then the annual failure probability is the probability that S = x and R < x for every value of x. So we have to compute the sum of the probabilities for all possible values of x:

$$P_f = \int_{-\infty}^{\infty} f_S(x) F_R(x) dx \tag{1}$$

This method can be applied when the strength and the load are independent of each other. In case of dependence, the failure probability can by determined by summation of the probability density of all the combinations of strength and load in this area.

$$P_f = \iint_{Z<0} f_{RS}(r,s) drds \tag{2}$$

In a real case the strength and the load in the limit state function are nearly always functions of multiple variables. For instance the load can consist of the water level and the significant wave height. In this case the failure probability is less simple to evaluate. Nevertheless with numerical methods like numerical Riemann integration (see **FLOOD*site*** report T07-08-01) and Monte Carlo simulation it is possible to solve the integral:

$$\begin{aligned}\mathrm{P}_f = \iint_{Z<0} \cdot\cdot \iint f_{r_1,r_2,\ldots,r_n,s_1,s_2,\ldots,s_m}(r_1,r_2,\ldots,r_n,s_1,s_2,\ldots,s_m) \\ \times dr_1 dr_2 \ldots r_n ds_1 ds_2 \ldots ds_m\end{aligned} \tag{3}$$

These methods which take into account the real distribution of the variables are called level III probabilistic methods. In the Monte Carlo simulation method a large sample of values of the basic variables is generated and the number of failures is counted. The number of failures equals:

$$N_f = \sum_{j=1}^{N} 1(g(\mathbf{x}_j)) \tag{4}$$

In which N is the total number of simulations. The probability of failure can be estimated by:

$$P_f \approx \frac{N_f}{N} \tag{5}$$

The coefficient of variation of the failure probability can be estimated by:

$$V_{P_f} \approx \frac{1}{\sqrt{P_f N}} \tag{6}$$

In which P_f denotes the estimated failure probability.

The accuracy of the method depends on the number of simulations. The relative error made in the simulation can be written as:

$$\varepsilon = \frac{\frac{N_f}{N} - P_f}{P_f} \tag{7}$$

The expected value of the error is zero. The standard deviation is given as:

$$\sigma_\varepsilon = \sqrt{\frac{1-P_f}{NP_f}} \tag{8}$$

For a large number of simulations, the error is normally distributed. Therefore the probability of the relative error E being smaller than $k\sigma_\varepsilon$ now equals $\Phi(k)$. For desired values of k and E the required number of simulations is given by:

Requiring a relative error of E = 0.1 lying within the 95 % confidence interval (k = 1.96) results in:

$$N > 400\left(\frac{1}{P_f} - 1\right) \tag{9}$$

The equation shows that the required number of simulations and thus the calculation time depend on the probability of failure to be calculated. For many structures in coastal and river engineering a relatively high probability of failure (i.e. a relatively low reliability) compared to structural elements/systems is calculated, resulting in reasonable calculation times for Monte Carlo simulation. The calculation time is independent of the number of basic variables and therefore Monte Carlo simulation should be favoured over the Riemann method in case of a large number of basic variables (typically more than five). In practice, however, it is possible to analyse the convergence of the probability failure during the course of the simulation. Specified convergence criteria are monitored during the simulation and when achieved the simulation is halted. Furthermore, the Monte Carlo method is very robust, meaning that it is able to handle discontinuous failure spaces and reliability calculations in which more than one design point is involved (see below).

The problem of long calculation times can be partly overcome by applying importance sampling. This is not elaborated upon here. Reference is made to the **FLOOD*site*** report T07-08-01.

If the limit state function (Z) is a sum of a number of normal distributed variables then Z is also a normal distributed variable. The mean value and the standard deviation can easily be computed. This is the base of the level II probabilistic calculation. The level II methods approximate the distributions of the variables with normal distributions and they estimate the limit state function with a linear first order Taylor polynomial, so that the Z-function is normal.

If the distribution of the Z-function is normal and the mean value μ and the standard deviation σ are known it is rather easy to determine the failure probability. By computing the reliability index β as μ divided by σ it is possible to use the standard normal distribution to estimate the failure probability. There are tables available of the standard normal distribution in the handbooks for statistics. The disadvantage of the level II calculation method is the inaccuracy in case of very non-linear limit state functions. However, the calculation speed is enormous and also fast insight is provided in the sensitivities of the random variables to the overall failure probability.

Considering all advantages and disadvantages of the available calculation methods, in **FLOOD*site*** task 7, it has been decided to use Monte Carlo simulation for failure probability calculation of large systems.

3 UNCERTAINTIES

Uncertainties are introduced in probabilistic risk analysis when we deal with parameters that are not deterministic (exactly known) but that are unknown instead, hence uncertain. Two groups of uncertainties can be distinguished (Fig. 2):

1. Natural variability (Uncertainties that stem from known (or observable) populations and therefore represent randomness in samples).
2. Knowledge uncertainties (Uncertainties that come from basic lack of knowledge of fundamental phenomena).

Natural variability cannot be reduced, while knowledge uncertainties may be reduced. Natural variability can be subdivided in natural variability in time and natural variability in space. Knowledge uncertainty can be subdivided in model uncertainty and statistical uncertainty; statistical uncertainty can be subdivided in parameter uncertainty and in distribution type uncertainty.

There is still discussion in literature about variability in space, for instance soil properties. On one hand, this spatial distribution of properties are mainly a case of lack of knowledge since there is only one realisation of the subsoil. On the other hand, it is practically impossible to reduce all uncertainties, resulting in a remaining (natural) variability.

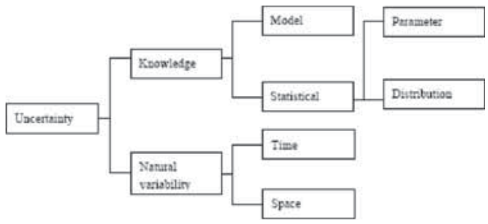

Figure 2. Classification of uncertainties.

One advantage of uncertainty classification is that it can be clearly seen which uncertainties might be reduced (knowledge uncertainties) and which ones not (natural variability). The influence of uncertainties on the reliability flood defences can be investigated in a probabilistic analysis by calculating the influence factors α_i's. These sensitivity coefficients show how much the variable contributes to the total probability of failure. In flood risk applications, the load variables seem to be dominant in case studies. These load variables embed natural variability and are therefore not reducible. This is in contrast with knowledge uncertainties which can be reduced by performing research (improving models), by gathering data or by expert judgement.

4 SYSTEMS ANALYSIS

A systems analysis can be carried out by constructing fault trees. Fault trees originate from the aircraft industry (NASA, 2002) and are subsequently used in chemical and computer industries. Fault trees are used to create insight in large complex systems with a large number of components and elements, like computers and aircrafts. The emphasis lies on identifying all possible causes (basic events) of all failure events and to assign failure probabilities to these basic events.

Generally a fault-tree can be divided into three layers. From bottom-up these layers are: bottom layer, intermediate layer[1], top layer.

The bottom layer exists of basic events or/and component failure. A basic event is for example the impact of a ship or other human failure, which can be quantified with a certain failure probability (i.e. $3.4 \cdot 10^{-6}$ per year). Component failure corresponds with failure of one of the components of the flood defence structure due to a certain failure (sub)mechanism. At this point the fault tree is fed with a (physical) model, describing the failure (sub)mechanism. Based on a model and data (i.e. soil parameters, hydraulic parameters, uncertainty, etc.) the failure probability of the component due to the (sub)mechanism can be determined. Subsequently, the result of the bottom layer is a set of failure probabilities.

The intermediate layer describes the several subsystems of the fault tree. In case of a flood defence structure these subsystems will correspond with the several failure mechanisms of the structure.

The top layer combines the failure probabilities of the several failure mechanisms into an overall failure probability of the structure.

[1] Depending on the complexity of the system several intermediate layers can exist.

When applying fault trees on flood defence structures, the emphasis lies more on 'Where to stop?' In order to find the right elaboration of a fault tree the following 'rules' could be helpful:

- Stop when a mechanism cannot be divided into sub mechanisms. Find the right model to describe the mechanism.
- Do not implement Basic Events or Component Failures when no data is available or when proper quantification is impossible.
- Do not implement events which are unlikely to occur.

In other words: 'Analyse no further down than is necessary to enter probabilistic data with confidence'

As mentioned above, Basic Events are tagged with a failure probability. This means that the events should be described as clearly and 'digital' as possible. 'Digital' means that there are two states: failure and non-failure. An example of a good description is: 'Drainage system failure'. An example of a bad description is: 'Groundwater flow behind structure' where the latter can be made 'digital' by using: 'Groundwater flow > critical flow velocity'.

A MOE is a Multiple Occurring Event and a MOB is a Multiple Occurring Branch. Both can occur within a fault tree. A MOE can for example be the water level exceeding a critical value or drainage system failure. MOE's and MOB's should be handled with care because they create dependency between two (sub)mechanisms. Dependencies should be taken into account when calculating the overall failure probability of a flood defence structure.

Cross references make fault trees more complex and can lead to circular-references. For example: Piping depends on Seepage. Seepage depends on too much settlement. Too much settlement depends again on piping. For simplicity sake cross-references should be minimized.

Fault trees should preferably be accompanied with pictures describing the underlying (sub) mechanisms, as shown in the appendix report of Task 7 (T07-08-02).

The systems analysis is supported by a Task 7 software package which comprises four main components:

- Structure specific fault trees—constructed externally within OpenFTA software (2000).
- Limit State Equations (LSEs)—comprised within a Dynamic Link Library (DLL) constructed from Fortran subroutines (developed and coded under **FLOOD*site***).
- Uncertainties on the input parameters—input through a spreadsheet interface (developed under **FLOOD*site***).
- Numerical integration—Monte-Carlo simulation implemented through C++ code using a Microsoft

Excel spreadsheet interface (**FLOOD*site*** development).

These components are depicted in Fig. 3. The primary outputs of the software are the annual probability of defence failure for a specific structure and a fragility curve for the structure.

A user friendly interface for the software package has been developed (Fig. 4). The interface is used to input information relating to:

- The name of the structure (determined from a drop down list which uses information from OpenFTA files with the pre-constructed fault tree. Initially, only one example (SheetPileWall structure) will be available until further structures are included in the Structure File tab.)
- Distribution functions for each parameter.
- Parameters for the distribution functions.
- The number of samples required for the Monte-Carlo simulation.

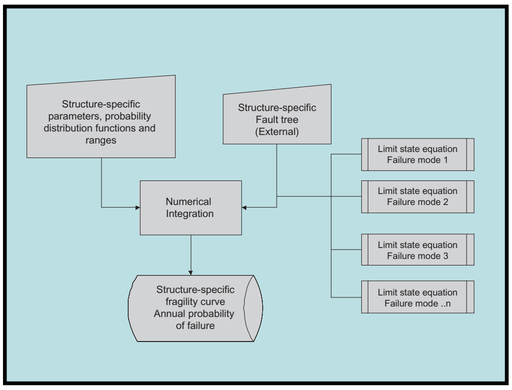

Figure 3. Components of the Systems Analysis.

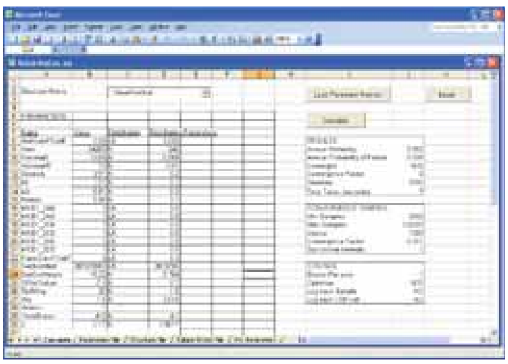

Figure 4. Interface of the software package for systems analysis.

- The required accuracy of the calculation (convergence).

More detailed information on using the interface and relating to these inputs are provided in the appendix report of Task 7 (T07-08-02), as well as information on extension of the tool to include additional structure fault trees and failure modes.

5 CASE STUDY

5.1 *Preliminary analysis*

Background on the case study is given in MLR (2001). Risk sources at the German Bight are resulting from storm surges in the North Sea associated with high water levels and storm waves at the flood defences. Typically, storm surges last not longer than 12 to 24 hours but may increase the water level considerably (up to 3.5 m in the North Sea). The interaction of normal tides (water level differences in the range of 1–2 m are normal in the North Sea region), storm surges, and waves is crucial for the determination of the water level at the coast. In addition, the foreshore topography plays a major role when determining the waves at the flood defence structure. In case of the German Bight the limited water depths over a high foreland will cause the waves to break and will therefore limit the maximum wave heights which reach the flood defence structures.

In case of the German Bight case study, flood defences comprise more than 12 km of dikes (grass and asphalt dike) and a dune area of about 2.5 km. The hazard analysis has however focussed on the dikes as the key flood defence structure since the dune belt is extraordinary high and wide and is regarded as significantly safer than the dike protection.

Before starting the probabilistic analysis the dike geometry and laser scan data have been used to define different sections of the flood defences. Criteria for distinction of different sections included the type of flood defence, its height, its orientation, the key sea state parameters e.g. water level and waves, and geotechnical parameters. Thirteen sections have been

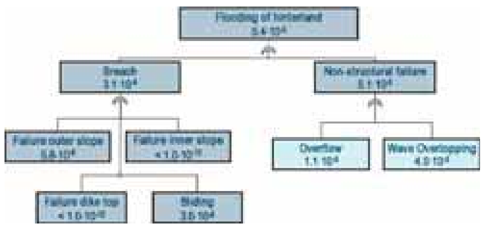

Figure 5. Typical fault tree for a dike section at "German Bight Coast".

identified using these criteria. Each of these sections is assumed to be identical over its entire length and hence will result in the same probability of failure.

The reliability analysis has used a full probabilistic approach as described in the previous sections of this paper, starting from the input parameters at the toe of the dike and applying the failure modes and fault trees which have been developed under **FLOOD*site*** for the specific type of flood defences. The result of this analysis is an annual probability of flooding of the hinterland for each selected dike section. These flooding probabilities were typically found to range from a probability of 10^{-4} to 10^{-6} which means a return period of flooding in the range of 10,000 or 1,000,000 years.

Based on the aforementioned probabilistic analysis the section with the highest probability of failure for breaching of the dike was taken as the section where a breach location was assumed.

5.2 *Detailed analysis*

The developed software tool for systems analysis was applied to the German Bight Coast case study and compared to the results obtained with a preliminary reliability analysis (see the appendix of the **FLOOD*site*** report T07-08-02). Figure 6 shows the variation in the dike crest elevation.

Failures in the past have not been reported, though considerable overtopping has been observed at a number of locations (Kortenhaus, 2003). An indication of relevant failure mechanisms is displayed in Figure 7.

Report T07-08-02 contains the overall fault tree that was applied to the German Bight Coast case study. The fault tree was constructed with the OpenFTA software. The database with failure mechanisms was connected to the chain of events in the fault tree: the wave driven erosion of the grass, clay cover layer and core. The duration of the three processes together was being compared with the storm duration. The erosion processes are then combined through an AND-gate. Such a solution underestimates the strength during a storm and will result in higher estimates of the probabilities of failure.

Outside slope instability and inside slope instability has not been included in the fault tree. Failure of the dike top also requires soil instability calculations and this was not included in the fault tree. However, previous analysis has shown that these failure mechanisms do not affect the probability of failure significantly.

The results of the preliminary reliability analysis of the German Bight Coast case study are listed in Table 1. The overall probability of failure for the different sections is given in Table 2.

The probability of failure was calculated with the reliability tool for section 1 and section 2, see Figure 7. Section 1 has a probability of failure of 5.56E-5 and section 2 has a probability of failure of 0.000279. These probabilities of failure are higher than the results obtained with the German Bight Coast case study site. The difference is explained firstly by the different arrangement of the fault tree of the reliability tool as compared to the scenario tree applied in

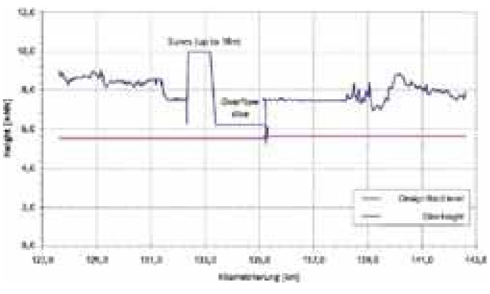

Figure 6. Height of coastal defence structures.

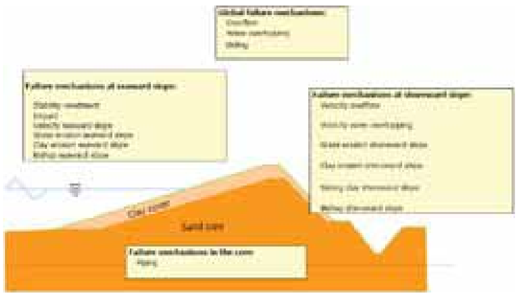

Figure 7. Indication of relevant failure mechanisms in the German Bight case study.

Figure 8. Screenshot of reliability tool for the calculation of the probability of failure of section 2 of the German Bight flood defences.

the German Bight Coast case study. A second explanation is a difference in limit state equations applied in the reliability tool as compared to the preliminary reliability analysis of the German Bight Coast, e.g. erosion or overtopping equations. A third explanation for the higher probability of failure is the application of different distribution functions and associated parameters. In some cases the value of the parameter as applied in the preliminary reliability analysis was not known and an assumption had to be made.

6 RECOMMENDATIONS

Several key areas for further research have been identified based on feedback from current and potential users of the reliability tool in regard to its present state utility and perceived limitations. These areas relate to:

- The inclusion of complex failure models in the tool, where explicit limit state functions cannot easily be defined or where the computational time required for evaluation of limit state functions is prohibitive.
- The extension of the tool to enable time dependent analysis of flood defences, whereby dynamic processes such as deterioration can be incorporated.
- The inclusion of a sensitivity analysis method for apportioning the uncertainty associated with flood defence failure to the variance in the resistance and stress input parameters.

These key areas are discussed further in the following sections.

6.1 *Complex failure models*

At present, all failure modes outlined in the **FLOOD*site*** Task 4 report (Allsop et al. 2007) have been included in the reliability tool, with the exception of those resulting in slip failure of embankments and dikes due to geotechnical instability. There are no simple, analytically solveable LSEs for these processes and, as such, numerical procedures must be used to evaluate defence failure. This significantly increases computational burden, making it difficult and time

Table 1. Overview of the results of the probabilistic calculations for all sections in the preliminary reliability analysis of the German Bight flood defences.

	Failure Probabilty Pf for sections [-]												
Failure modes	1	2	3	4	5	6	7	8	9	10	11	12	13
Overflow	2,00E-06	4,80E-07	2,00E-06	4,80E-07	4,70E-07	3,60E-05	2,99E-04	9,80E-04	9,10E-05	1,12E-05	2,50E-05	2,70E-05	2,60E-05
Overtopping	6,00E-06	5,00E-06	5,00E-06	1,80E-05	4,10E-05	1,26E-04	6,51E-04	1,90E-03	1,40E-04	4,00E-05	1,67E-04	1,59E-04	1,58E-04
Sliding	2,00E-06	4,00E-06	3,00E-06								2,00E-06	3,00E-06	2,00E-06
Impacts	9,57E-04	5,70E-04	4,67E-04								6,00E-05	1,25E-04	1,30E-04
Velocity wave run-up	1,51E-01	1,12E-01	1,20E-01								3,63E-02	6,11E-02	6,19E-02
Bishop outer slope	0,00E+00	9,60E-05	1,00E-05				not analysed				0,00E+00	0,00E+00	0,00E+00
Velocity overflow	2,00E-06	2,00E-06	3,00E-06								2,00E-06	2,00E-06	2,00E-06
Velocity overtopping	4,50E-05	6,00E-06	2,30E-05								1,60E-04	1,38E-02	3,84E-04
Bishop inner slope	3,45E-04	0,00E+00	0,00E+00								1,00E-06	0,00E+00	0,00E+00
Piping	3,00E-06	6,00E-06	2,00E-06								3,00E-06	3,00E-06	4,70E-07
Sceanrios													
SC I	0,00E+00	0,00E+00	8,30E-05								2,70E-05	9,80E-05	1,03E-04
SC II	2,80E-05	1,30E-05	9,36E-04								1,25E-03	4,60E-03	4,70E-03
SC III	0,00E+00	0,00E+00	0,00E+00								0,00E+00	1,80E-05	0,00E+00
SC IV	0,00E+00	0,00E+00	0,00E+00								0,00E+00	0,00E+00	0,00E+00
SC V	0,00E+00	0,00E+00	0,00E+00				not analysed				0,00E+00	0,00E+00	0,00E+00
SC VI	0,00E+00	0,00E+00	0,00E+00								0,00E+00	2,15E-03	0,00E+00
SC VII	0,00E+00	0,00E+00	0,00E+00								0,00E+00	0,00E+00	0,00E+00
SC VIII	0,00E+00	0,00E+00	0,00E+00								0,00E+00	0,00E+00	0,00E+00
SC IX	1,50E-05	2,00E-06	7,00E-06								2,00E-06	0,00E+00	1,50E-05
SC X	0,00E+00	0,00E+00	0,00E+00								0,00E+00	0,00E+00	0,00E+00
SC XI	0,00E+00	0,00E+00	0,00E+00								0,00E+00	0,00E+00	0,00E+00

Table 2. Overview of the overall probability of failure for each flood defence section in the preliminary reliability analysis.

	Failure Probability Pf for sections [/yr]												
Failure modes	1	2	3	4	5	6	7	8	9	10	11	12	13
Overall probability of failure	1.0E-5	9.5E-6	1.8E-5	1.8E-5	4.1E-5	1.6E-4	0	2.9E-3	2.3E-4	5.4E-5	2.0E-4	2.3E-4	1.9E-4

consuming to calculate the probability of failure within a Monte Carlo Simulation (MCS) approach. For the same reason, simplifying assumptions have been made for many of the other LSEs included in the reliability tool, where this was not seen to significantly hinder the accuracy of the resulting reliability estimates. However, there may be instances where it is desirable to explicitly account for the more complicated processes that will potentially result in defence failure (e.g. sliding of embankments, which depends on an adverse combination of several factors). Thus, the exclusion of complex failure processes may present a significant limitation to the reliability tool.

To overcome this problem, it is possible to use fitted response surface models as surrogates, or "emulators", of more complex process models. Artificial neural networks (ANNs) are a type of data-driven model that highly suitable for use as emulators due to their ability to model any continuous nonlinear function to arbitrary accuracy and their rapid run times. Therefore, further research will focus on the development of ANN models for use as emulators of complex flood defence failure processes within the reliability tool. An initial case study will involve the development of an ANN model to represent the failure of a flood wall in the New Orleans flood defence system, which breached during Hurricane Katrina in 2005, primarily due to sliding. A finite element model of this flood wall has since been developed and applied within a probabilistic MCS failure analysis approach (RajabaliNejad et al. 2007). Using this approach, a very high failure probability was correctly estimated under the loading conditions resulting from Hurricane Katrina; however, this calculation was very time consuming due to the complex nature of the finite element model. On the other hand, an ANN model, fitted to the responses of the finite element model, will provide a mathematical function that overcomes the need for finite element analysis; thus, resulting in an efficient model for analysing such failures using the reliability tool.

6.2 *Time dependent processes*

The reliability tool currently represents flood defence reliability as a snapshot in time. However, time-dependent processes in the hydraulic climate (e.g. water levels and wave conditions), as well as the behaviour of flood defence properties (e.g. crest levels, vegetation, erosion), can lead to time-dependent defence reliability. The incorporation of such processes within a reliability analysis allows the explicit consideration of processes that may reduce (e.g. deterioration due to history of loading) or increase (e.g. growth of vegetation) the structural stability of flood defences in time. This can be extremely important when considering future flood defence reliability and may allow emergent failure processes to be revealed (e.g. the deterioration of a structure may trigger seemingly unimportant failure mechanisms). Therefore, the ability to incorporate time-dependent processes within the reliability tool should be a key area for future research.

Whilst not yet included in the reliability tool, Buijs et al. (2008) developed a methodology for incorporating time-dependent processes in a flood defence system reliability analysis. Using this approach, the probability that the lifetime of a flood defence is less than period *t* can be calculated, as follows:

$$Z(t) = g(X_1, \ldots, X_{i,t}, \ldots, X_n) \tag{10}$$

where $X_{i,t}$ introduces time-dependency in the limit state function $Z(t)$. The methodology is used to conduct an uncertainty analysis of the flood defence properties driving or contributing to the time dependent process by directly modelling the nature of the process and any dependencies with other processes.

Incorporation of time-dependent processes within the reliability tool would require the inclusion of a method to randomly sample from the joint probability of dependent parameters, where the cross- and autocorrelations between the parameters are preserved.

6.3 *Sensitivity analysis*

In the past, reliability-based methods, such as First- and Second Order Reliability Methods (FORM and SORM, respectively), have often been used for failure probability calculation in order to overcome the computational burden historically associated with MCS. However, MCS is usually more accurate than these reliability methods and with increasing computer power, the computational requirements of MCS are no longer prohibitive for reliability analysis involving rather simple LSEs, such as those included in the reliability tool. However, a limitation of MCS-based reliability analysis, in comparison to the FORM and SORM approaches, is that it does not provide the contribution to the probability of failure from each input random variable, as the latter approaches do. Therefore, an important area for further research is the extension of the reliability tool to include a sensitivity analysis method that can be used in conjunction with the MCS-based reliability analysis to determine input importance.

Variance-based sensitivity analysis provides a method for apportioning the total variation in Z to the input variables $X_1, \ldots, X_n$, which can be useful for determining whether Z can be stabilised by better controlling the inputs and which inputs are most important in this process. For example, VBSA can be used to assess whether variation in the erosion endurance of vegetation cover significantly contributes to the variation in *Z* for a given flood defence; in which case, better control over the type of vegetation may significantly improve the structural reliability of the defence. An advantage of VBSA over other sensitivity analysis approaches is that, when using a MCS-based reliability analysis method, relatively little additional computational effort is required, since the MCS input samples are used in the calculations.

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