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Representing fragility of flood and coastal defences: Getting into the detail

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ABSTRACT: Investment planning and decision-making for performance-based management of flood and coastal defence assets requires representation of their fragility. Generalised defence fragility representations by major asset types may be used for broad-scale systems analysis, but when making local asset management decisions more structure-specific representations become important. The FLOOD*site* and UK FRMRC projects have developed a Reliability Tool to generate structure-specific fragility curves, based on a reliability analysis of multiple potential failure modes linked by fault trees. To assist practicing engineers to understand, accept and start using fragility curves, work has been carried out under the UK PAMS (Performance-based Asset Management System) project to clarify the connection between deterministic and probabilistic approaches, and to demonstrate ways (including use of the Reliability Tool) of estimating or calculating defence fragility starting from conventional engineering practice. The paper will present conclusions from this work, giving some example comparisons between deterministic design and full reliability analysis.

1 BACKGROUND

Flood risk managers need to target spending and management intervention in areas of greatest flood risk, whilst seeking to maximise the overall return on investment and achievement of other targets. This targeting needs to be informed by an improved understanding of the overall risk, the attribution of risk to individual assets and the likely change in risk that would result from an engineering intervention. In turn, these issues can only be addressed through an improved understanding of the behaviour of a single asset and the asset system as a whole.

A key component in this approach is the derivation and representation of asset fragility in the form of fragility curves. Fragility curves quantify the relationship between the loading on an asset and the conditional probability of failure of the asset given that loading. In UK flood risk management, they are typically determined by a probabilistic reliability analysis. The fragility curves enable the performance of defences to be taken into account in a system-wide flood risk analysis (Sayers *et al*, 2002).

Currently, the generalised fragility curves generated during the development of the RASP methodology and utilised within the national or regional flood risk assessment models (see e.g. Gouldby *et al,* 2008) represent the only nationally available consistent dataset on defence fragility. Although these nationally available fragility curves differentiate some 60 defence types and utilise the local loading conditions and some of the geometry of a specific defence, they are based on simplified representations of the overall defence condition, limited local data and a limited number of failure modes. Fragility needs to be representative of local conditions if reliable policy and decision making is to be achieved at the local scale. When wishing to make reliable local asset management decisions, these more accurate site/structure-specific representations of fragility become critical.

In generating these more specific representations it is also important to achieve an understanding of fragility curves as a risk-based tool in the eyes of UK flood defence practitioners as they move to a whole-life and risk-based approach to asset-management. In principle, well-established deterministic methods also look at the relation between loading and probability of failure (although in a very implicit way), so there is an opportunity to establish a link. At the same time, a better link could improve the appreciation of fragility concepts among designers, which could stimulate them to make more use of the rational, but non-traditional concept of designing to the whole continuum of possible loading conditions, instead of to one fixed standard of protection.

Deterministic methods typically calculate a safety factor for one given configuration of loading and strength. In deterministic design, various structural arrangements are analysed to arrive at the one that best fulfils all objectives: meeting the required standard of protection, and at the same time balancing costs, whole-life considerations and secondary objectives.

Achieving a better representation of fragility is not straightforward, however, and will need careful thought to ensure that:

- The development of the fragility curve is transparent and is not perceived by practitioners as a black art. Where either expert judgment or modelling is used, the evidence upon which the fragility analysis is based will need to be clearly stated and recorded—so that it can be challenged (and improved) with time.
- The development of the fragility curves maintains consistent estimates of fragility across all asset types. For example it will be important to ensure that as the condition of the asset deteriorates the fragility increases in all cases and that the correct ordering between types is maintained.
- The resultant improved fragility assessments are useable within the context of an analysis of flood risk—including the data used to derive them, the representation of the defence type and the axis used to express the load.
- The curves are sufficiently accurate to support the decisions that they are supposed to support.
- The approach to development of the curves is realistic about data availability.

As part of the ongoing Thames Estuary (TE) 2100 studies, the issue of accurately representing fragility has emerged as particularly important to ensure reliable outputs from the RASP RFSM system model, the RASP RFSM system model being a PAMS-type systems analysis model (HR Wallingford, 2007; Gouldby *et al*, 2008; Simm *et al*, 2006) permitting assessment of flood risk, its distribution and attribution to defence lengths along the Thames Estuary. A particular feature of the Thames defences is the dominance of large composite structures with unusual failure modes not properly represented by the (HLM+) generic fragility curves, with their own site specific combinations of failure modes. Site-specific studies are underway to provide more representative fragility curves for the TE2100 more detailed modelling, the significance of which will be mentioned later in this report.

The FLOODsite (Task 7) and FRMRC (WP4.4) Projects have led to the development of a sophisticated flood defence reliability calculator. This prototype software tool (van Gelder, 2008) facilitates the construction of fault trees, selected from a range of over 50 different limit state equations (Allsop *et al*, 2007) and enables reliability calculations (i.e. fragility curve generation) to be undertaken on a site specific basis, if site specific data is available.

This paper explains how this tool can be used along with other methods to develop fragility curves starting from deterministic design practice.

2 DETERMINISTIC DESIGN (LEVEL 1 PROBABILISTIC)

The Level 1 probabilistic design approach is that which is adopted in most European Standards. The essence of the standards is that a certain representative value of the strength or resistance (R_{rep}) is divided by a factor and that the representative value of the load (S_{rep}) is multiplied by a factor, for which the following must apply:

$$\frac{R_{rep}}{\gamma_R} > \gamma_S\, S_{rep}$$

where the factors γ_R and γ_S are known as partial safety factors.

The representative loads and strength values used in a Level 1 design or analysis are calculated in a way that reflects the fact that there is statistical variability in both the loading and strength input data to engineering calculations. It sets design values a fixed number of standard deviations away from the mean:

$$R_{rep} = \mu_R + k_R\sigma_R$$

$$S_{rep} = \mu_S + k_S\sigma_S$$

where k_R is negative and k_S can be positive or negative.

This last point is crucial for designers to remember when moving to the probabilistic design thinking for fragility curves. Not only do means and standard deviations for loading and strength have to be assessed, but care has to be taken not to confuse traditional representative load or strength information with mean values. As the last two equations capture, they are not the same: the mean value being near the centre of the data cloud whereas the old representative load tends to be near the upper or lower bound of the data cloud.

3 FRAGILITY CURVES

3.1 *Introduction*

The *fragility* of a structure is defined as the probability of failure conditional on a specific loading (Casciati and Faravelli, 1991). The concept of fragility has been widely used in reliability analysis in other industries to characterise structural performance across a range of imposed loads. These applications can be divided into three main categories:

1. Fragility curves based on empirical data with, as main requirement, sufficient failure data available of different loading conditions (e.g. earthquake engineering/mechanical engineering).
2. Fragility curves based on expert judgement (e.g. nuclear industry/USACE flood defence).
3. Fragility curves based on structural reliability methods employing limit state functions.

The last category allows the use of conventional physical process-based models in the absence of failure data. This approach is usually preferred in coastal and flood defence reliability analyses given the infrequency of the extreme design loadings.

The concept of fragility was first postulated for use in flood risk management in the USA by the US Army Corps of Engineers (1993). However it was not progressed into application to a full systems analysis in the US. Instead it was first introduced to flood risk assessments in Europe in the United Kingdom to represent the link between the likelihood of defence response (pathway) given different hydraulic loading conditions (source) (Dawson & Hall, 2002; HR Wallingford, 2003).

Whilst fragility may appear to be a challenging concept to understand, it is relatively easy to compare it with conventional design approaches. Figure 1 illustrates deterministic design, in which the assumption is made that the probability of failure is zero until the design load event is reached, at which point the probability switches to 1. The risk of failure under design loading at the Ultimate Limit State (ULS) is minimised by employing partial safety factors on loading and strength.

In reality of course, at the design load, the probability of failure is in fact not zero but a small number. As Figure 2 illustrates, as the load rises above the design condition, the probability of failure rises and only after considerable extra load has been applied does it actually approach 1.0. Similarly, there is a small but significant probability of failure at conditions below the design loading.

In practice there are many uncertainties in the estimation of the values of the conditional probability of failure (probability of failure given a certain load) that make up fragility curves. Hence it is usual to give upper and lower bounds on these curves rather than a single value.

Since the concept of fragility was introduced into risk assessments in the UK, it has been successfully applied in a number of case studies and in national flood risk assessment (Hall *et al*, 2003; Gouldby *et al*, 2008).

Risk assessments in the UK are based on the source-pathway-receptor (s-p-r) model (Sayers *et al*, 2002). Within this model the consequences, given

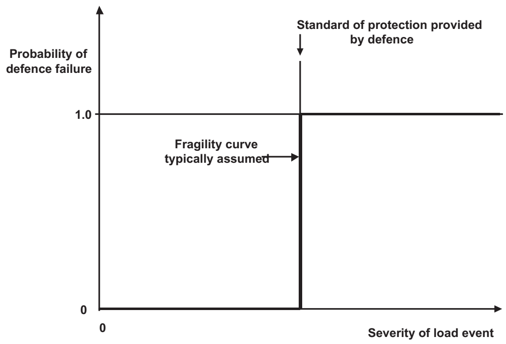

Figure 1. Fragility curve according to deterministic design (Sayers & Meadowcroft, 2005).

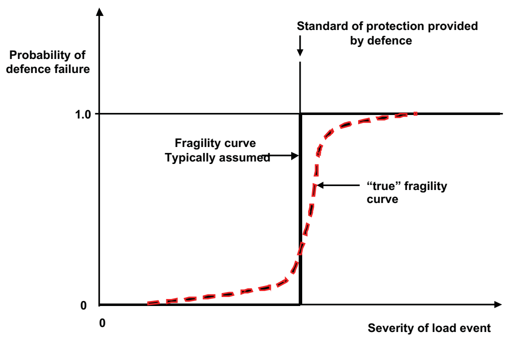

Figure 2. True fragility curve compared with deterministic design (Sayers & Meadowcroft, 2005).

a number of different possible responses of the pathway, are determined, which in turn are dependent upon different source conditions. The generally applied definition of risk is that it equals the likelihood of an event, times the undesired consequences of that event. In flood risk assessments this amounts to the following:

$$\{\textit{Magnitude of flood risk} \mid \textit{flooding scenario}\}$$
$$= \mathrm{P}(\textit{failure of defence} \mid \textit{hydraulic loading conditions})$$
$$\times \{\textit{damage} \mid \textit{flooding scenario}\}$$

where the hydraulic loading conditions represent the source in the s-p-r model.

The pathway of the hydraulic loading conditions into the floodplain can be via (a) wave overtopping/overflow, (b) failure of the flood defence which initiates a breach formation process, and/or seepage through or under the structure. The breach formation process of the defence given certain loading conditions results in flooding of an area of the floodplain. The consequences of this flooding scenario are expressed in terms of damage to the receptors in the floodplain.

A fragility curve in flood risk assessments represents the probability of breach of the defence given a set of loading conditions and therefore represents (along with the probability of overtopping) part of the likelihood of water flowing from source to receptor in the s-p-r model for flood risk assessments.

3.2 *Loading parameters in fragility curves*

In principle, the *hydraulic* loading parameter for a structure fragility curve can be selected as any parameter of interest. However, for consistency of the subsequent use in systems-analysis (PAMS-type) models (such as the RASP RFSM being used on the TE2100 project), it is beneficial to standardise on a common approach. This takes:

- *water level* as the key parameter for *fluvial defences* and
- *overtopping unit discharge*, for *coastal defences*.

The latter has been taken as it is a convenient simplification to amalgamate the multivariate loadings (wave height & period, water level etc) into a single (univariate) loading and thus avoids having fragility surfaces lying over a multi-parameter loading space.

There might be situations where it is desirable to express a failure of a defence or component within the flood and coastal defence system not as a function of these hydraulic loading conditions but of other loading (source) conditions. Ideally, in this case, it should be aimed to construct fragility curves given the loading conditions that relate directly to the consequences involved with failure. However, in practice, for flood risk systems analysis it is normally necessary to keep to a consistent view of loading. This may lead to situations in which the estimate of probability of failure is more-or-less constant across the range of conceivable hydraulic loadings (i.e. the fragility curve is effectively flat.)

3.3 *Standard fragility curves currently being used in broad-scale UK models*

Generalised fragility curves (Defra/EA, 2005) have been generated for use in UK national flood risk assessment studies as part of the RASP (Risk Assessment for Strategic Planning) methodology (Hall *et al*, 2003). These curves have been based on a *classification* of linear flood defences, which at the highest level identifies the following main defence types:

- Embankment or sloping seawall.
- Slope protection against coastal erosion.
- Vertical wall structures (e.g. sheet piles, concrete slabs, masonry walls).
- Beaches (sand, gravel).

This system leads to a total of 61 defence types. For each defence type fragility curves (best estimates and upper and lower bounds) have been developed for each of five nationally-defined Condition Grades. Of these, only Condition Grade 1 can be directly compared to the initial structural design condition to which most designers work. Condition Grade acts as a kind of 'label' for each fragility curve, even though the way it is normally assessed (visual inspection) means that it cannot take account of all the structural processes which may be relevant to failure.

Other assumptions embedded within the standardised fragility curves developed for the systems analysis methodology are:

- Only failure modes due to high water levels are considered; this is consistent with the definition of fragility as being a function of hydraulic load. However, this assumption does not deal with the issue of low water failure modes and the fact that 'waterward' failure of defences can lead to overtopping during the next high water period.
- All curves are based on a standardised defence height of 1.5 m and on representative failure modes.

3.4 *Generation of site-specific fragility curves*

To generate site-specific fragility curves for flood defences, given detailed information of their structural and foundation properties and loading conditions, it is necessary to do the following:

1. Define the overall function(s) of the flood defence.
2. Systematically identify and analyse all relevant failure modes likely to lead to flooding, and the interaction between these failure modes. In this first stage

analysis, conventional deterministic approaches can be helpful to eliminate unrealistic failure modes.

3. Identify an appropriate "model" to represent each failure mode(s). In many cases this model will be some kind of (ultimate) Limit State Equation (LSE). In some cases (e.g. slip failure) this will not be possible and use of models, such as finite element models, may be necessary. (The procedure in the latter case is explained in the next section.) Having identified the Limit State Equation or model recast it in reliability form:

   Z (*reliability*) = R (*strength*)—S (*non-hydraulic loading*)—S (*hydraulic loading*)

   where *R* represents the gathering together of all terms or parameters which relate to the strength of the structure and *S* represents the gathering together of all terms or parameters which relate to the magnitude of the loading.
4. Produce a schedule of the engineering parameters feeding into the LSEs, including defining the width and form of the uncertainty bands around each parameter.
5. Prepare fault trees that specify the logical sequence of all possible failure mechanisms leading to the failure of the defence.
6. Perform a series of reliability analyses under a series of different hydraulic loading conditions. Each analysis for a given loading condition comprises of a series of Monte Carlo simulations (across the uncertainty bands for each input parameter). Failure arises in a particular case when the combinations of parameter values in the limit state function Z gives a value for Z which is less than or equal to zero. The probability of failure for that loading is then the number of times when the simulation gives Z as less than or equal to zero divided by the total number of simulations.
7. Repeat Step 6 for an appropriate series of different hydraulic loadings, and from the results draw a fragility curve.

3.5 *The reliability tool*

To make the above process easier, under FLOODsite Task 7, a flexible software 'Reliability Tool' was developed to analyse the reliability of flood defences. The tool includes a total of 72 failure modes represented as simple Limit State Equations (LSEs), a flexible fault tree component, and a probabilistic failure analysis component based on Monte Carlo simulation (MCS). It is applicable to foreshores, dunes and banks; embankments and revetments; walls; and point structures, and accounts for hydraulic loading due to water level difference across a structure; wave loading; and lateral flow velocities.

The user interface of the Reliability Tool is provided via a MS Excel spreadsheet. For a given flood defence structure, values must be supplied for each of the parameters required by the relevant LSEs. A value may be fixed or specified as a statistical distribution with associated parameters. Using a Monte Carlo technique, random sample values are generated according to the specified distributions. For each sample, the fault tree is evaluated calling subroutines for the associated LSEs and using the sample values.

To generate fragility curves using the Reliability Tool, the hydraulic loading conditions are specified as fixed variables. These are then varied systematically, with a failure probability calculated for each value of loading considered, leading to the generation of fragility curves.

3.6 *Dealing with a failure mode not included in the reliability tool*

Some failure modes are not yet included in the Reliability Tool. Although there is ongoing work to add more modes, some processes in some structures require a more specific representation (e.g. via finite element modelling). In these cases, the Reliability Tool should be used first to generate a fragility curve for all failure modes for which it holds an LSE.

For the failure mode not included in the Reliability Tool, repeated runs of the structural models (e.g. for slope stability for embankments) should be carried out using the known variability of input parameters. (Some models have a 'Monte Carlo' operating mode to facilitate this.) This exercise will yield the probability of failure for a given hydraulic loading. The process can then be repeated for other hydraulic loadings and a fragility curve for this failure mode built up.

An overall fragility curve can then be generated by combining the fragility curve for this additional failure mode with the fragility curve generated by the Reliability Tool. The method for combining these analyses is to combine the event probability of failures, conditional on the hydraulic load that are output from the respective models, using de Morgan's law:

$$\Pr(f) = 1 - \{[1 - \Pr(f_r)] \times [1 - \Pr(f_s)]\}$$

where *fr* and *fs* denote the event probabilities of failure, conditional on the loading level, from the reliability calculator and the structural model, respectively.

The latter combination method is only valid if *fr* and *fs* are independent of one another: in other words, if the additional failure mode is not dependent on any of the other failure modes already analysed in the Reliability Tool.

4 COMPARING DETERMINISTIC AND PROBABALISTIC APPROACHES

4.1 *Evidence from recent analysis of failures during the floods in England during summer 2007*

4.1.1 *Overall assessment*

The flood events in June and July 2007 in England tested large numbers of flood defences and can therefore be used as a validation event for our understanding of defence performance. With regard to design methods and probabilistic approaches, analysis of the failures can give an indication of the conditional probability of failure because of the large lengths of defences that were tested.

Evaluation by the Environment Agency indicated that just over 1,000 km of linear defences were tested by the floods. Approximately half of this length was overtopped (525 km). Of all these defences, only four embankments actually breached during the events, over a total length of about 50 m. This means that out of the total defence length that was overtopped, about 0.01% breached (or about 0.2% in terms of the number of assets). There are only limited references for the probability of breach that should be expected at 'design loading' or above. However, Dutch flood defences are designed to a 'safety philosophy' which states that the probability of breach up to design loading has to be less than 10% (TAW, 1998). Set against that background, the percentage of breaches during the Summer floods is very low.

Figure 3. Location of assets for Summer floods performance analysis.

Figure 4. Breach at Auckley during Summer 2007 floods.

4.1.2 *Analysis of individual failures*

The Environment Agency commissioned a specific review of the technical performance of the defences (Royal Haskoning, 2008). This review analyses the performance of the defences that breached, overtopped or were severely tested by high water levels (see Figure 3). Information was collated on site, from existing datasets and from anecdotal evidence, aiming to determine loading, strength and failure modes.

When examining the failures of individual assets, it was found that at least three of the four breaches (see example in Figure 4) happened while the water level was significantly below the crest (and this is uncertain for the fourth and final breach). This means that these three embankment breaches were caused by geotechnical failure modes. The analysis shows that the breaches were not caused by overall poor quality of design or condition, but by local irregularities. These irregularities may be visible (such as the presence of foxholes or disruptive vegetation) and hence captured in the condition grade. But they can also be invisible and related to the embankment material or to the subsoil. The analysis did not find a strong correlation between condition grade and breach. Taken as whole, however, uncertainty about the presence of internal irregularities is probably a significant factor driving the small probabilities of failure in the fragility curves in the part where water levels are below crest level.

In contrast to these isolated low-water-level geotechnical failures, of the 500 km of defences that were overtopped, all (possibly but one) were able to withstand significantly more than the nominal overtopping which grassed embankments are expected to withstand. This finding reflects some of the latest research on resistance of grass against overtopping (predominantly by waves) in the Comcoast project; results of field tests suggest that good quality non-reinforced grass can withstand significantly higher discharges than thus far expected, and for significant durations (Royal Haskoning, 2007).

For all analysed sites, the situation at the moment of breach was compared with the existing generic fragility curves. The analysis generally shows that for the breached defences, the generic fragility curve shows a very small probability of breach (see example in Figure 5), while for the defences that overtopped but did not breach, the generic fragility curve predicts significant probabilities of breach.

4.2 *Comparisons between geotechnical factor-of-safety approaches and fragility curves*

4.2.1 *Embankments*

For the simplified standard embankments upon which the existing generalised fragility curves are based, a deterministic analysis was carried out using the geotechnical analysis package MSTAB. Geotechnical analysis typically determines a Factor of.

Safety and then uses a maximum value (typically around 1.3 but to some extent at the discretion of the designer) to determine what is acceptable for design. The analysis was carried out for a wide range of geometries and assumptions for geotechnical characteristics of subsoil and fill, in order to model the range of situations represented by the generalised fragility curves. For normal freeboards for fluvial defences (of the order of 0.3 m below the crest) the Factor of Safety did not fall below 1.3.

A similar exercise was conducted for one of the TE2100 exemplar sites, a wide multi-bermed embankment, using the finite element package SLOPE-W. The resulting variation in geotechnical Factor of Safety is shown in Figure 6.

The corresponding fragility curve associated with that geotechnical slip failure mode is shown in Figure 7. The fragility curves were derived using the

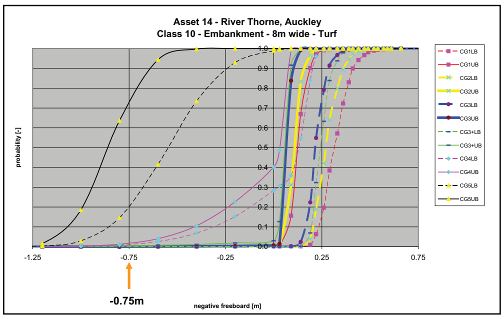

Figure 5. Conditions at Auckley during Summer 2007 floods at time of breach plotted on standard fragility curves.

SLOPE-W package in its probabilistic mode, using means and standard deviations for all parameters.

Note that the Factors of Safety shown in the *curves* in Figure 6 are those calculated using mean values of parameters. However, for comparison Factors of Safety calculated using conservative lower bound values on strength parameters and upper bound values on weight (loading) parameters are shown for the design water level.

From a comparison between these graphs the following can be inferred:

1. In a situation (water at extreme design level) where conventional analysis using conservative parameters would have assessed the Factor of Safety of 1.0, the conditional probability of failure (probability of failure given loading) is between 5% and 10%.
2. In a situation (water at crest level) where conventional analysis using mean parameters would have assessed the Factor of Safety as 1.0, the conditional probability of failure (probability of failure given loading) is about 50%.

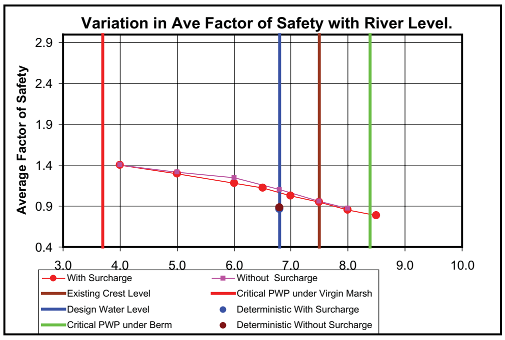

Figure 6. Variation in Factor of Safety with water level for a wide multi-bermed embankment in the Thames Estuary.

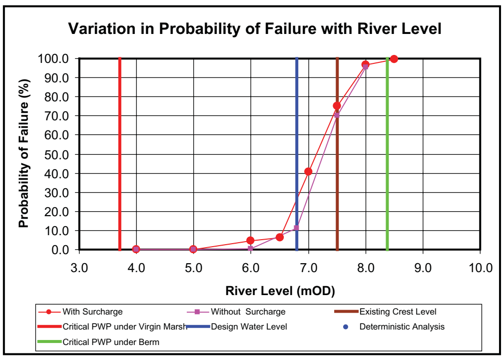

Figure 7. Fragility curve for the same wide multi-bermed embankment in the Thames Estuary shown in Figure 4.

Both of these conclusions are intuitively correct. The first is consistent with the idea that probabilities of failure under design extreme loading should not exceed 10%, which is captured in design code philosophies and also embodied in the Dutch design philosophy. The second suggests that if mean values of all parameters are used, the condition at which a factor of safety of about 1.0 is achieved should have a probability of failure of about ½. In other words, if a structure is just on the tipping point of failure, the probability that it will fail might be expected to be equal to the probability of it not failing.

4.3 *Comparisons between deterministic overtopping design and fragility curves*

For overtopping, good practice design does not work with Factors of Safety, but with simple empirically based design rules that are known to provide a sufficiently conservative structure.

First, steady state overtopping can be examined. A number of parameters are explicitly taken into account in deterministic analysis which are assumed as fixed parameters in the fragility curves (such as landward slope angle and loading duration). On the other hand, the fragility curves take into account other additional factors (such as residual strength of clay cores). Figure 8, derived from a deterministic analysis, shows the location on the fragility curve of the range of loading values that would be acceptable in design (as a function of loading duration and slope angle). The location for poor grass cover is generally consistent with the fragility curve for Condition Grade 4 and the location for good grass is generally consistent with the Fragility Curve for Condition Grade 1. In both cases the design range is slightly to the left (the conservative side) of the fragility curves; were the fragility curves to be adjusted to the right to reflect the results of the latest COMcoast research, the design range would be further to the left of the fragility curve (as would be expected).

For wave overtopping of coastal structures, good practice design is based purely on overtopping discharge, which is also the parameter on the horizontal axis of the fragility curves for coastal assets. Figure 9, derived from the deterministic analysis, shows the location on the fragility curve of the loading value that would be acceptable in design. As expected it is in a conservative position to the left of the fragility curve.

Following this exercise, a site-specific fragility curve using the Reliability Tool discussed in Section 3.3, was constructed for the steady state embankment overtopping case (see Figure 8 above). This time the parameters describing landward slope angle and quality of grass cover, for example, were not given conservative values, but were assigned probabilistic distributions to capture the ranges in the parameters appropriate to the deterministic analysis. A fixed storm duration of 3 hours was assumed. To construct the fragility curve, the magnitude of freeboard over the embankment was systematically increased, whilst reliability was calculated for each value of freeboard considered. This produced the curve shown in Figure 10. Also shown in Figure 10 is the range of critical freeboard obtained in the deterministic analysis for a storm duration of 3 hours. This range accounts for grass cover of poor to good quality and a landward slope of 1:2 to 1:4.

From Figure 10 it can be seen that the range of critical freeboard for erosion under deterministic design matches well with the area on the fragility curve where the probability of failure is rising rapidly. It can be concluded that a fragility curve for steady state overtopping tailored to specific conditions gives a good match with an equivalent deterministic design approach using mean value parameters.

A further step in analysis (not carried out in this project) would be to carry out deterministic calculations for a range of negative freeboards of existing/acceptable velocity and compare the specific velocity

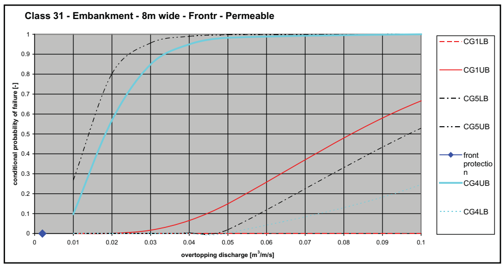

Figure 8. Comparison of fragility curve with good practice design—overflow, class: river embankments.

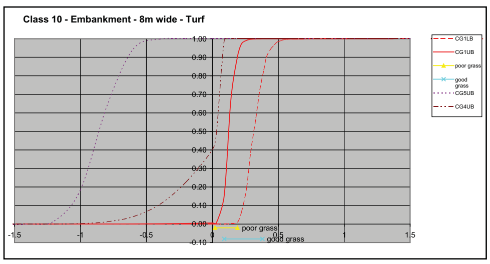

Figure 9. Comparison of fragility curve with good practice design—overtopping, class: coastal embankments.

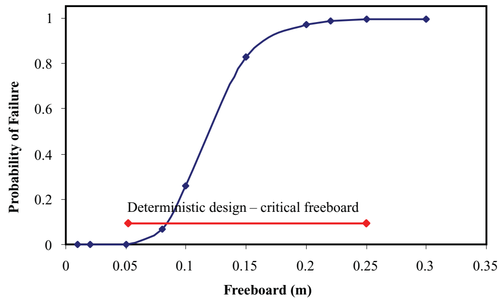

Figure 10. Comparison of site-specific fragility curve derived using the Reliability Tool to good design practice.

values with the fragility curve probabilities derived using the Reliability Tool. This would start to provide guidance on relating deterministic 'critical velocity' values to conditional probabilities of failure.

5 CASE STUDY: DEVELOPMENT OF FRAGILITY CURVES FOR THAMES DEFENCES

As part of the development of a regional flood-risk system model for feasibility stage of the Thames Estuary 2100 studies on the long-term flood defence strategy for the Thames Estuary, site-specific fragility curves were needed in order to ensure more reliable decision-making on options. This work provided a first full-scale demonstration of the methods set out in this paper.

The approach adopted was to select 15 exemplar sites around the Thames Estuary which were representative of the majority of the structure types. They included embankments and 'hard' defences, the latter mostly of concrete, brickwork and/or sheet piling. Development of the fragility curves was relatively straightforward because conventional engineering studies had already been carried out to assess the state of the existing defences in order to explore options for future maintenance and renewal or defence raising.

For each structure analysed, the way the engineering understanding fed into the step-by-step process (described in Section 3.4 above) for developing the fragility curves was as follows:

1. *Define function(s) of the flood defence*.
   This was a joint exercise between the engineering and reliability analysis teams.
2. *Identify and analyse all relevant failure modes*.
   Engineering input here was vital. Quick deterministic analyses enabled a number of possible failure modes to be removed from the reliability analysis on the basis that they only offered negligible probabilities of failure.
3. *Identify Limit State Equations (LSE) or models for all failure modes and recast them in reliability format*.
   In most cases the LSEs identified were already included within subroutines in the Reliability Tool. A notable exception was that of geotechnical slip failure. In this latter case, a standard finite element geotechnical package (SLOPE-W) was used in probabilistic format.
4. *Prepare schedule of engineering parameters and their uncertainties*.
   Here engineering parameters which had already been developed for the deterministic design thinking were re-used. However, careful thought was required to represent the mean and standard deviations of each parameter correctly, taking care not to confuse traditional representative load or strength information with mean values.
5. *Prepare fault trees that specify the logical sequence of all possible failure mechanisms leading to the failure of the defence*.
   This was a relatively straightforward process, given the careful engineering thinking that had been used at Step 2. Care was required however to ensure that the fault tree reflected any dependencies between failure modes.
6. *Perform reliability analyses for a range of hydraulic loadings to generate a fragility curve*.
   Here the Reliability Tool was generally used and this ensured that any dependencies between failure modes were taken into account. However, the results of the WSLOPE analysis for geotechnical slip failure had to be added afterwards, using de Morgans' law as described in Section 3.3.2 above. The proforma for the embankment exemplar site shows the combination of a fragility curve from the Reliability Tool (overtopping induced erosion failure mode) and the WSLOPE fragility curve.

6 TRANSLATION OF SITE-SPECIFIC FRAGILITY CURVES TO A FULL SET OF DEFENCES IN A SYSTEM

As described in Section 1, a key reason for developing fragility curves is to facilitate a system wide assessment of defence performance and thereby to prioritise management action. For this purpose, it is necessary to have fragility curves for every defence length, irrespective of their imagined performance or significance.

Whilst it may be satisfactory for initial thinking to make use of national generic fragility curves, once serious decision-making commences these curves will have to be improved. In the TE2100 studies, it was clear that the national fragility curves were particularly unrepresentative for a number of reasons:

– The height of the defences was greater than the national average.
– The structural forms of the defences were often composite (e.g. sheet piling with an embankment).
– The underlying geology was complex, with soft clays overlying a layer of water bearing gravels connected to the river and exhibiting high piezometric pressures during flooding.

However, the time and effort of developing fragility curves for every defence can be considerable. If faced with the challenge of improving curves, one option is only to develop new curves for selected defences representative of the remainder. The process involves selecting a number of 'exemplar' defences. (In the case of TE2100, fifteen structures were selected to represent the defences downstream of the existing Thames Barrier). Each exemplar defence should have a structural form and strength, associated foundation geology and hydraulic loading environment which is representative of a number of other defences in an asset system. The fragility curves for this exemplar defence should then be developed for all potential condition grades and can also be used for all similar defences in the asset system.

Whilst this approach is a significant simplification, it will be a rational and achievable step forward from using generic national curves, and the resulting with curves can be significantly different. The subsequent systems analysis can still reflect further detail in local differences, for example, in terms of variation in crest levels, condition grade and depth of postulated breaches.

Determining which exemplar defence is representative of which other defences can be a time-consuming exercise. Approaches can include:

– Comparing basic information about the structural form of the assets (e.g. RASP type, plans, sections and photographs, crest level) on an individual asset by asset basis.
– Identifying a mapping between the exemplar defence and the national asset classification (RASP type) allocated to remaining defences of similar type.

7 CONCLUSIONS

1. Fragility curves are an essential part of understanding and managing the performance of defences from a risk perspective. They help to inform:
   – The analysis of the systems-wide behaviour of defences, helping not only in the overall assessment of benefits for management intervention but also in identifying those defences which should be prioritised for that intervention;
   – An understanding of the conditional probabilities of failure of defences given flood event loadings of different magnitudes;
   – An understanding of the effect of intervention works on asset reliability.
2. Whilst generic, and thus approximate, fragility curves for different asset types can be used for broad scale analysis, site-specific curves should be developed and used whenever possible when prioritising local management interventions.
3. Where sufficient information is not available to follow a process for developing full fragility curves, it is possible to identify two points on a fragility curve from deterministic design thinking:
   – At the normal design point (design extreme water level or overtopping event) if conventional (conservative) approaches to determining engineering parameters and solutions, the probability of failure will typically be between 1% and 10%.
   – If mean values for engineering parameters (without conservatism bias) are used to identify a hydraulic loading condition at which the factor of safety against failure is about 1.0, then at this loading condition the probability of failure will be about 50%.

   If desired these two points could be used to check and/or adjust generic fragility curves as a first crude estimate, without having to resort to any reliability analysis.
4. However, generating full site specific fragility curves is an achievable task, so long as a clear engineering understanding of the performance of the structure concerned has been developed. This must include:
   – Identification of all the key failure modes and their interrelation, ruling out failure modes which generate negligible probabilities of failure.
   – Identification of mean values and statistical distributions (in most cases standard deviations will be sufficient) for all load and strength parameters affecting the key failure modes.
5. The generation of site-specific fragility curves then follows a clearly defined process set out in this paper.
6. The generation of site-specific fragility curves can be supported by appropriate reliability tools, such as that developed under FRMRC1 and FLOOD*site* and now being beta-tested. Standard software packages, such as finite element packages for soil slope stability, may also contain useful routines to enable fragility curves to be developed for specific failure modes.
7. Development of fragility curves for real sites, such as the TE2100 exemplar sites has demonstrated

that with sound engineering input the resultant curves are believable and consistent with traditional engineering practice.

8. Uncertainty in developing reliable fragility curves can be reduced by:

– local and historical knowledge of structures and their performance held by asset managers.
– careful engineering investigations of loadings, structural state and ground conditions.

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