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Guide to risk assessment for reservoir safety management

Volume 2: Methodology and supporting information

Report – SC090001/R2

Flood and Coastal Erosion Risk Management Research and Development Programme

The Environment Agency is the leading public body protecting and improving the environment in England and Wales.

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Our work includes tackling flooding and pollution incidents, reducing industry's impacts on the environment, cleaning up rivers, coastal waters and contaminated land, and improving wildlife habitats.

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Miranda Kavanagh

Director of Evidence

Executive summary

It is a considerable challenge to ensure acceptable performance and to manage risk from dam assets in the short to longer term through physical interventions to maintain, repair, improve or replace assets, while avoiding unnecessary expenditure. The wide variety in dam types and forms and physical settings further complicates the task. Within the context of this complex setting, the concepts of risk and performance provide dam managers with a consistent framework to analyse and understand the critical components of their dam, and the system within which it sits, and to target effort in further data collation, assessment or physical intervention appropriately.

A scoping study conducted by the Environment Agency in 2009 (SC070087/R1) established the need to update the *Interim Guide to Quantitative Risk Assessment for UK Reservoirs*, originally published in 2004 to provide a tool for the management of reservoir safety. It was recommended that this update should include a review of the risk management framework and that this and the procedures developed should meet a wider range of reservoir owner/undertaker and industry needs as well as meshing with current UK government flood risk assessment policy and practice.

Reservoir safety management is a process of managing the risk of an uncontrolled release of the contents of a reservoir. This new document has sought to explain and guide the user through the steps of the risk informed approach to reservoir safety management, providing an introduction and explanation of basic concepts through a detailed application of the methods and appropriate links to other reference documents and useful guidance.

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- Atkins Ltd (Andy Hughes)
- HR Wallingford (Michael Wallis and Alexandra Topple)
- Jacobs (John Gosden)
- RAC Engineers and Economists (David Bowles)
- Samui (Mark Morris)
- Sayers And Partners (Paul Sayers)
- Stillwater Associates (Alan Brown)
- United Utilities (Keith Gardiner)

The risk assessment methodology was tested on a series of dams in England and Wales. These were conducted by engineers at Jacobs, Atkins Ltd, and HR Wallingford with assistance from the reservoir owners who contributed funding, and engineers who provided data and information on the reservoir dams including. Key contributors include:

- Bristol Water
- Canal and River Trust
- Dŵr Cymru Welsh Water
- Northumbrian Water
- Severn Trent Water
- United Utilities

The guidance has also benefited from feedback and comments from many others during its development including inspecting engineers, supervising engineers, reservoir owners/undertakers and managers, representatives of the National Farmers Union, the Country Land Owners Association, the English Golf Union, the Angling Trust, and other professional partners and consultants. In an open process, three freely accessible consultation workshops were held and well attended. Those who participated receive our particular thanks. All the debates and comments have helped to develop the guidance into a stronger document that should enable reservoir safety risk assessments to be conducted in a robust and consistent way, founded on good practice.

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1 Introduction

The purpose of this guide is to provide a tool for the management of reservoir safety using risk assessment methods.

This guide (especially the Tier 1 analysis) has been designed with the application of risk assessment to be undertaken by a variety of users in mind. It should also be considered applicable by owners of non-regulated reservoirs. This is in line with good practice in the UK and overseas to undertake a risk assessment of reservoirs in a periodic safety review. Further risk assessment may be justified as a result of such a review or at other stages in the reservoir risk management process.

The level of detail included in such an assessment should depend on the level of confidence that is required to support various types of reservoir safety decisions.[1] This can be expected to vary with the level of risk posed by a specific reservoir and the inspecting engineers and owners/undertakers requirements (where applicable) for confidence and defensibility in supporting their decisions.

Societal concerns, including the perspectives of other stakeholders, such as the population at risk, should also be considered. It is important to identify the lack of knowledge (and thus the uncertainty) that exists about the factors that determine the performance of a reservoir, and the risks that these pose. This guide therefore uses a tiered approach to risk assessment. The different tiers (three) in this approach provide tools and methods that are proportionate in terms of level of effort required, detail considered and confidence in their outcomes.

This framework and its associated tools and methods provide an approach that allows the reservoir owner or undertaker, inspecting engineer or supervising engineer to better understand and evaluate reservoir safety risk in a structured way. This in turn allows for risk-based decision-making that can reduce risks to people, the environment, the economy and the owner/undertaker, while maintaining an important reference to accepted good practice.

1.1 The risk assessment process

Stakeholders and engineers involved in dam safety have different objectives. An adaptable process of risk assessment that includes different methods to assess various aspects of the reservoir system is therefore useful. The methods outlined in this guide allow the user to assess reservoir safety risks in either a qualitative (Tier 1) or quantitative (Tiers 2 and 3) manner, depending on user needs.

A risk assessment should commence with a clear definition of its purpose. This includes an identification of the decisions that it will inform and the information that those decisions require.

In outline, the risk assessment entails the process shown in Figure 1.1:

a) Pre-assessment preparation

- Establishing the context and objectives of the risk assessment and collecting available information

[1] This approach is referred to as a 'decision-driven' (NRC 1996) approach to determining the level of sophistication.

b) Risk identification

- Looking at event and failure scenario[2](s) through failure mode identification

The purpose of risk identification is to identify what might happen or what situations might arise that could affect the safe operation of the reservoir. This includes identifying the causes and source of the risk (hazard in the context of physical harm), events, situations or circumstances which could have a material impact upon reservoir safety, and the nature of that impact. Thus risk identification process should consider loads on the dam, potential modes of failure and the types of consequences of failure to include in the risk analysis.

c) Risk analysis

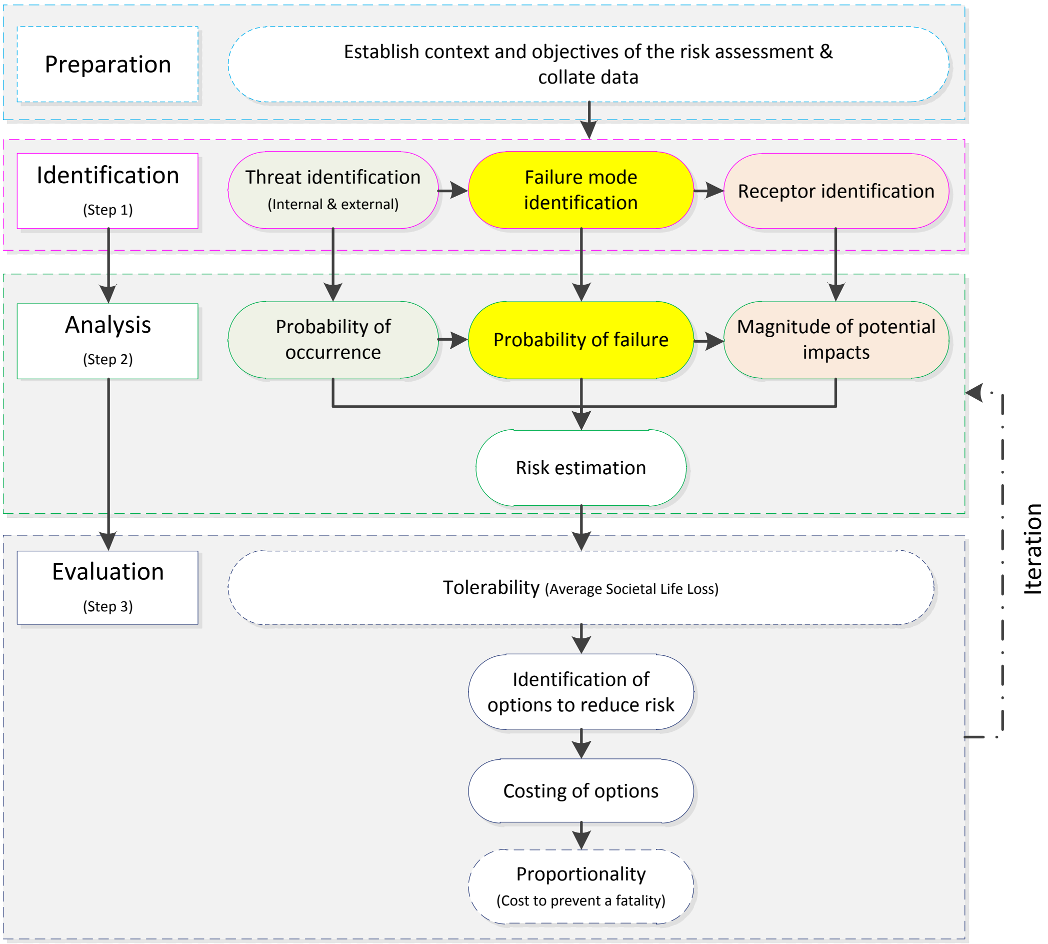
- Looking at the probability or likelihood that an event will occur (loads, failure, dam break and inundation) and considering consequence scenario(s) through impact assessment, building on the results of the risk identification analysis

d) Risk evaluation

- Looking at the tolerability of the risks calculated in terms of good practice, ALARP (as low as reasonably practical), the cost benefits of options to reduce risks and other owner/undertaker or stakeholder specific considerations

These stages are common in all three tiers of risk assessment.

Figure 1.1 The risk assessment process



[2] Event and failure scenarios are the chain of actions that lead to a reservoir failure and an uncontrolled release of its content.

The layout of this guide reflects the stages of the risk assessment process as shown in Figure 1.2.

Figure 1.2 Overview of the components of the risk assessment

Risk Assessment Framework

Pre-assessment preparation

Establish:
- the context
- the objectives of the assessment
- the risk building blocks

Collate appropriate site information and data
- Collate & review existing report, drawings etc
- Consult with Supervising Engineer

Step 1	Risk Identification	Step 1a	Failure modes identification (FMI)
		Step 1b	Identify potential consequences
		Step 1c	Review; scope risk analysis
Step 2	Risk Analysis	Step 2a	Likelihood of failure due to internal threats
		Step 2b	Likelihood of failure due to external threats
		Step 2c	Dambreak and flood routing
		Step 2d	Consequence analysis
		Step 2e	Determine level of risk
		Step 2f	Review outputs
		Optional	Estimate range of uncertainty
Step 3	Risk Evaluation	Step 3a	Review tolerability of risk assessment
		Step 3b	Review options to reduce risk
		Step 3c	Proportionality
		Step 3d	Other considerations
		Step 3e	Review and make recommendations

1.2 How to use this guidance document

This document (Volume 2) should be considered in conjunction with Volume 1. It has two parts:

- Part 1 provides a step by step guide to each tier of risk assessment.
- Part 2 provides additional information in support of the methods in each tier. This is essential background for engineers who apply the methods.

The first part of this document contains the risk assessment methodologies for Tiers 1, 2 and 3. A Tier 1 risk assessment might be undertaken routinely for any reservoir as an initial data collection and assessment exercise. Where risks are already known to be high, one might proceed directly to a higher tier, which builds on the Tier 1 level of analysis. Much of the effort needed to conduct a Tier 1 risk assessment is typical of what might be expected to be undertaken routinely when performing a Section 10 inspection for regulated reservoirs.

Tier 1 only evaluates a limited number of common threats and failure modes. Where other threats and failure modes are considered likely to be significant at a particular dam either a higher tier should be used, or the Tier 1 method extended to these.

A Tier 2 level of analysis provides a base quantitative estimate of reservoir risk. This would be undertaken when risk issues have been identified at a Tier 1 level, or are already known, and the risk needs to be quantified to support appropriate management actions.

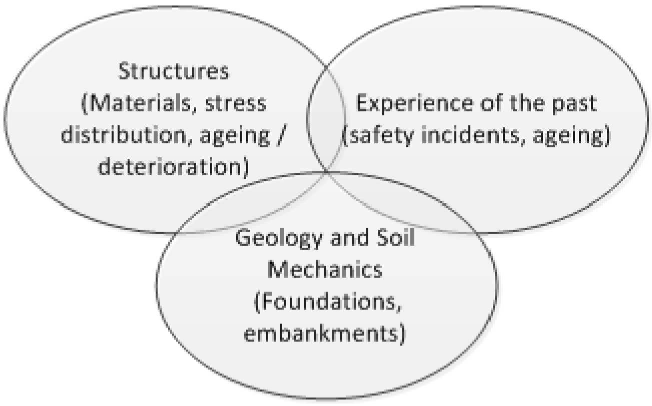
The Tier 3 level of analysis introduces more complex methods for assessing risks and interdependencies between processes. This level of analysis entails the use of more complex models and methods for determining potential processes and the integration of these analyses within the overall assessment of risk. The extent to which the analyses may be undertaken varies, and will depend upon the magnitude of the risk posed by the reservoir and the level of confidence required to support decision-making.

The effort required for analysis in each of the tiers is proportionate to the risk. A Tier 3 level of analysis is most likely to be undertaken where an earlier Tier 1 or 2 analyses has identified high potential risks and the magnitude of these risks justifies the effort required to analyse and reduce the uncertainties around the prediction, so supporting management decisions and risk reduction actions. For more information on the basis of the tiered approach see Chapter 15.

It is important to remember that the purpose of this guide is to assist users in the management of the safety of their reservoir dam by encouraging them to think critically, rather than a being a prescriptive methodology which should be applied without thought.

The factors governing the analysis of likelihood of failure are indicated in Figure 1.3. Although detailed numerical analysis of likelihood of failure is possible and assists in understanding sensitivities, the problem needs to be defined correctly if the output from such analysis is to be meaningful – hence the importance of good pre-assessment preparation. The potential pitfalls of overreliance on detailed quantitative analysis are spelt out by both Vaughan (1994) and by the Health and Safety Executive (HSE) in its description in *Reducing Risk, Protecting People* (R2P2) (HSE 2001, paragraph 93). Thus the output should always be reviewed critically by engineers experienced with dams. If applied without thought, situations can occur where the output is misleading.

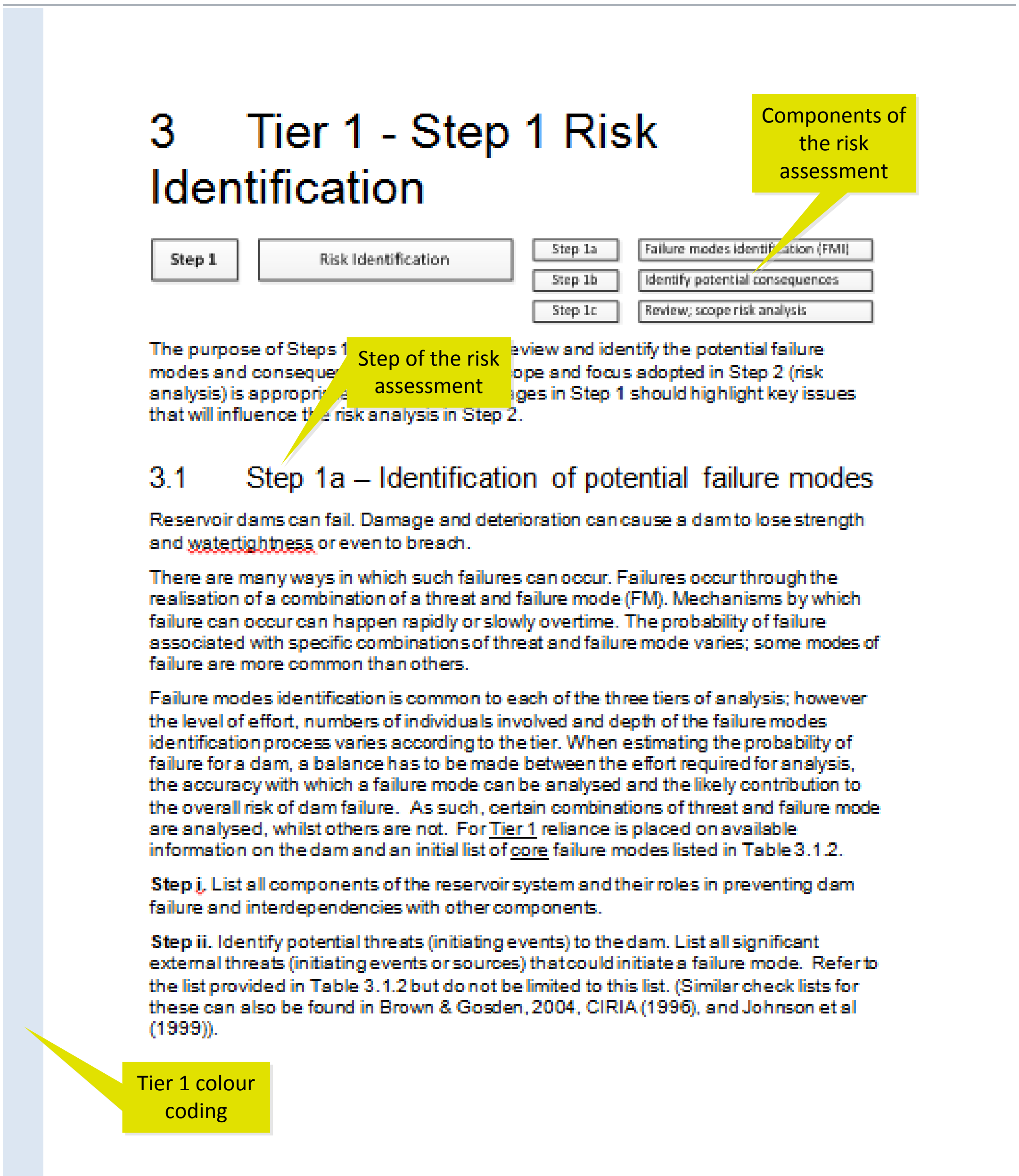
Figure 1.3 Building blocks for assessment of likelihood of failure



1.3 Finding your way around the document

There are various ‘navigation’ aids to assist you in finding your way around this document. Each tier is colour coded. For Tier 1 the pages feature a blue edge column (as in the example shown in Figure 1.4), Tier 2 a brown column, and Tier 3 green. Signposting to supporting guidance in Part 2 of the document is also provided by text situated within these coloured columns.

Figure 1.4 Example of Tier 1 page highlighting ‘navigation aids’



3 Tier 1 - Step 1 Risk Identification

Step 1	Risk Identification	Step 1a	Failure modes identification (FMI)
		Step 1b	Identify potential consequences
		Step 1c	Review; scope risk analysis

The purpose of Steps 1 [illegible] eview and identify the potential failure modes and conseque [illegible] ope and focus adopted in Step 2 (risk analysis) is appropri [illegible] ges in Step 1 should highlight key issues that will influence t [illegible] e risk analysis in Step 2.

3.1 Step 1a – Identification of potential failure modes

Reservoir dams can fail. Damage and deterioration can cause a dam to lose strength and watertightness or even to breach.

There are many ways in which such failures can occur. Failures occur through the realisation of a combination of a threat and failure mode (FM). Mechanisms by which failure can occur can happen rapidly or slowly over time. The probability of failure associated with specific combinations of threat and failure mode varies; some modes of failure are more common than others.

Failure modes identification is common to each of the three tiers of analysis; however the level of effort, numbers of individuals involved and depth of the failure modes identification process varies according to the tier. When estimating the probability of failure for a dam, a balance has to be made between the effort required for analysis, the accuracy with which a failure mode can be analysed and the likely contribution to the overall risk of dam failure. As such, certain combinations of threat and failure mode are analysed, whilst others are not. For Tier 1 reliance is placed on available information on the dam and an initial list of core failure modes listed in Table 3.1.2.

Step i. List all components of the reservoir system and their roles in preventing dam failure and interdependencies with other components.

Step ii. Identify potential threats (initiating events) to the dam. List all significant external threats (initiating events or sources) that could initiate a failure mode. Refer to the list provided in Table 3.1.2 but do not be limited to this list. (Similar check lists for these can also be found in Brown & Gosden, 2004, CIRIA (1996), and Johnson et al (1999)).

1.4 Dam types to which this guide applies

This guide applies to all types of embankment dam and to gravity structures. Gravity structures include both concrete or masonry dams and also service reservoirs. The guide does **not** apply to buttress or arch dams.[3] Detailed guidance on risk analysis for arch dams can be found in the US Bureau of Reclamation’s online *Dam Safety Risk Analysis Best Practices Training Manual* (Reclamation 2009–2011). The methods in this guide can be applied to buttress dams if they are considered as gravity structures. Definitions are given in the Glossary.

[3] Although the guide does not provide for the quantification of probability of failure for arch and buttress dams, the failure modes identification (FMI) process described on the guide can be used for these structures.

PART ONE – The method

2 Preparing to assess the risk

Pre-assessment preparation	Establish: the context the objectives of the assessment the risk building blocks Collate appropriate site information and data Collate & review existing report, drawings etc Consult with Supervising Engineer

Before commencing the risk assessment it is important to set the context, determine its objectives and identify the risk guidelines to be used.

2.1 Establishing the context

The level of detail of the assessment will depend on the level of detail and confidence that is required to support various types of reservoir safety decisions. The types of safety decisions that are to be made should therefore be identified before commencing any risk assessment to determine the appropriate level of detail to usefully inform the decision-making process (see ‘Selecting an initial tier of risk assessment’, Volume 1, section 4.6).

Such considerations should set the risk assessment in context and ensure that the information provided adequately informs the decisions to be made.

Factors to consider include:

- user needs and potential benefits (see Volume 1, section 2.3)
- the specific decisions to be made
- degree of confidence and defensibility required

2.2 Establishing the objectives

The objectives of the risk assessment must be clearly set out before starting the assessment. They should reflect the purpose and content of the assessment and may include a need to:

- improve understanding of the potential failure modes for a reservoir
- improve understanding of the magnitude of the consequences of dam failure
- provide information on the likely extent of flooding and the number of people that would need to be evacuated, in the event of a dam failure
- consider the effect of changes in inspection and monitoring programmes
- assess options and alternatives to reduce the risk
- optimise the control of risk during any remediation process
- estimate residual risk remaining after remedial works and identify appropriate risk control strategies
- provide reassurance to others – especially those at risk of flooding

For a portfolio of reservoirs, such objectives can help to:

- compare and prioritise risks arising from the portfolio of reservoirs (in order to prioritise actions)
- provide information for corporate risk management and insurance purposes
- demonstrate due diligence in the management of the reservoir safety programme

The risk assessment tools can assist in these objectives by providing:

- a systematic means of identifying modes of failure.
- a transparent record of the risk assessment
- a means of quantifying, prioritising and monitoring risk management actions

2.3 Establishing the building blocks of the risk assessment

The risk analysis methods in each tier involve the same building blocks – scenarios of inundation and estimation of the consequences.

2.3.1 Scenarios of inundation

When preparing for the risk analysis it is important to consider which loads or ‘events’, or combination of loads, could lead to failure and inundation. This can be done through qualitative or quantitative description of ‘scenarios’ of failure and release of water, and an estimate of the associated extent and depth of inundation. This includes consideration of:

- external threat (or loading event)
- internal threat
- dam failure modes
- inundation depth, extent and other characteristics

2.3.2 Estimation of consequences

Similarly it is necessary to determine the consequence scenarios which include, for example, criteria such as the exposure of people at the time of inundation, with or without prior warning. The consequences for given event or failure scenarios (determined above) can then be accounted for or calculated. This includes consideration of potential impacts on:

- people
- critical infrastructure
- economic activity
- the environment
- cultural heritage

The method of risk evaluation provided in this guide is based on the Tolerability of Risk (TOR) framework adopted in HSE's R2P2 guidelines (HSE 2001), that is, reducing risk to life to as low as reasonably practicable. However owners/undertakers may wish to consider evaluation by other metrics such as likely loss of income to their business through structural damage and associated loss of revenues.

Owners/undertakers may also wish to consider the risk of affected stakeholders bringing private actions against them for losses and damages as a result of dam failure. Damage to reputation may also be a significant factor to consider. Such consequences could be added to the methodology. There are no accepted 'tolerability' benchmarks for types of consequences other than risk to life. However owners/undertakers may wish to consider such concerns for their business risk assessment or as a factor in the prioritisation of works within their portfolio of reservoirs.

Risk scenarios to be used in the risk analysis should be determined based on:

- objectives of the risk assessment (section 2.2)
- information available (see section 2.4 on site information and data)
- level of risk assessment (the tier)
- type of reservoir
- hydraulic nature of its catchment (for example, rapid run-off)
- nature of the reservoir system (if in a cascade for example)
- type of dam and its construction (earth, concrete, masonry)
- nature of the land use and occupancy of the valley below the reservoir

2.4 Collate the site information and review the data

Information and data required for the risk assessment should be identified and collated from various sources. The objective is to build up a good picture of the construction and current condition of the dam. Headings would normally include those in Table 2.1.

Table 2.1 Indicative types of information about dam

	Intrinsic condition (materials used and quality of the build)			**Current condition** (see section 2.4.7)	
	Materials and dimensions	**Detailing**	**Function of element,** for example, essential to dam safety (preventing release of reservoir), or operational?	**Evidence of possible onset of dam failure**	**Any maintenance requirements**
Example for embankment dam	Type of fill/ foundation Level of top of core, crest, spillway	Interface between concrete structures and fill			Wave erosion damage Safety of access Painting pipework
Examples for concrete gravity dams		Spacing and type of vertical contraction/ expansion joints in perimeter wall			
Examples for service reservoirs		As for concrete gravity dams Roof – wall connection			Leakage leading to poor bacteriological conditions

2.4.1 Definition of the reservoir system and its components

For the purpose of conducting a reservoir safety risk assessment it is proposed that a broad definition of the reservoir system is required to include the following components:

- The reservoir, including surrounding hillsides, the dam(s) and their abutments and foundations, all appurtenant structures electromechanical equipment, all instrumentation, communication systems and any other natural or man-made physical features that are relevant to the safe operation of the reservoir.
- Operating, maintenance, monitoring, surveillance and inspection procedures, including any manuals and the logic and any software needed to implement any automated or remote control of reservoir operations, including inflows and discharges and information such as inflow flood forecasts, management systems and communications and decision protocols, upon which safe reservoir operation depends.

- Human factors, including operations and maintenance, monitoring and surveillance, supervision and inspection, and all management aspects of the owner/undertaker or operating organisation upon which safe reservoir operation depends.

2.4.2 Site information and data collation

The amount of information available on a dam varies enormously from site to site. A Tier 1 assessment offers the simplest approach for a risk assessment and should be able to be conducted based upon a review of available information. Note that the information required for a Tier 1 qualitative risk assessment is typical of what might be expected to perform a Section 10 inspection.

A Tier 2 or 3 analysis, however, may well require additional data or analysis to support the quantitative assessment of risk.

2.4.3 Collation and review of existing reports, drawings and so on

The sources of information available to support the risk assessment will usually include some or all of the following:

- prescribed form of record under the Reservoirs Act, where available (includes key dates and dimensions)
- inspecting engineer's reports under Section 10 of the Reservoirs Act 1975
- supervising engineer's statements under Section 12 of the Reservoirs Act 1975
- monitoring reports
- instrumentation records
- valve operating records
- water level records
- performance history
- as built drawings
- land use maps

It is particularly useful to identify previous studies that may have highlighted existing deficiencies. Such information should be used to inform the identification of failure modes for the risk assessment. However, the distinction between deficiencies and factors that can lead to a risk of dam failure must be clearly understood.

2.4.4 Consultation with the supervising engineer

For regulated reservoirs, consultation with the supervising engineer is considered to be an essential part of the process of gathering information for a risk assessment, including failure modes identification, and analysis and development of the inputs to the risk analysis.

2.4.5 Consultation with the owner and other stakeholders

A risk assessment for a reservoir may be instigated by an owner/undertaker due to non-technical concerns raised by other stakeholders. These concerns should be recognised by the risk assessor through consultation with the owner/undertaker. Particular issues may determine the level of analysis required to satisfactorily address them.

It is good practice for the assessor to consider the approach to Step 3 (Risk evaluation) before conducting the risk assessment to ensure that the right tier of assessment is chosen[4] and the right level of analysis is conducted in order to generate enough information to the right level of detail (and confidence) to answer the questions posed and to satisfy the requirements of the owner/undertaker and other stakeholders.

2.4.6 Engineering judgement

It is essential to realise that risk assessment is an aid to engineering judgement. It should not be the sole basis for the commissioning of works required to improve the safety of the dam.

An important part of the proper preparation for the risk assessment is in compiling existing engineering assessments for the dam. From these it should be determined whether the dam has been assessed against published standards (such as floods (ICE 1996) and seismic vulnerability (Charles et al. 1991, ICE1998), or accepted good practice, and if so, did it meet these published standards and/or are there potential or confirmed existing deficiencies that need to be addressed.

Where a dam is judged deficient when measured against published standards but an ALARP analysis shows that further works are disproportionate in cost, engineering judgement should be used to inform the briefing given to decision makers. Some commentary on the issue is included in Chapter 21. The judgement should also include consideration of issues such as whether the published standards reflect current best practice, or are dated and due for revision, and uncertainties in the ALARP analysis.

2.4.7 Condition assessment (optional)

If there is not a currently valid condition assessment with sufficient information available to inform the risk assessment, consider performing a new condition assessment.

The main benefits of a condition assessment include:

- a better understanding of the physical elements forming the dam, and their detailing, (the last Section 10 report could supplement/support this)
- recording the condition of all elements of the dam for future reference
- informing the assessment of likelihood of failure (may be conducted as part of the risk analysis)

The condition assessment needs to record the condition of all elements of the dam and reservoir system. Identify evidence for any internal and external threats, and potential failure modes. An example template for condition assessment is shown in Table 16.2.

[4] See Table 4.2 in Volume 1 guide to assist in choosing the correct tier of assessment.

2.5 Involving personnel familiar with the dam

The risk assessment should be carried out utilising the knowledge of personnel familiar with the dam such as the inspecting engineer for example (especially in Step 1a – failure modes identification). For regulated reservoirs this is normally performed in a workshop session, with the supervising engineer (or other personnel familiar with the dam) describing the various elements of the dam using photographs and reports.

For more detailed studies a site visit may also be helpful though, at all tiers, the workshop/interview with the supervising engineer is normally the most effective means of ensuring the risk assessment makes full use of site specific information.

3 Tier 1 – Step 1 Risk identification

Step 1	Risk Identification	Step 1a	Failure modes identification (FMI)
		Step 1b	Identify potential consequences
		Step 1c	Review; scope risk analysis

The purpose of Steps 1a, b and c is to review and identify the potential failure modes and consequences such that the scope and focus adopted in Step 2 (risk analysis) is appropriate for the site. The stages in Step 1 should highlight key issues that will influence the risk analysis in Step 2.

3.1 Step 1a – Identification of potential failure modes

Reservoir dams can fail. Damage and deterioration can cause a dam to lose strength and watertightness, or even to breach.

There are many ways in which such failures can occur. Failures occur through the realisation of a combination of a threat and failure mode (FM). Mechanisms by which failure can occur can happen rapidly or slowly overtime. The probability of failure associated with specific combinations of threat and failure mode varies; some modes of failure are more common than others.

Failure modes identification (FMI) is common to each of the three tiers of analysis. However, the level of effort, numbers of individuals involved and depth of the FMI process varies according to the tier.

When estimating the probability of failure for a dam, a balance has to be made between the effort required for analysis, the accuracy with which a failure mode can be analysed and the likely contribution to the overall risk of dam failure. As such, certain combinations of threat and failure mode are analysed, while others are not.

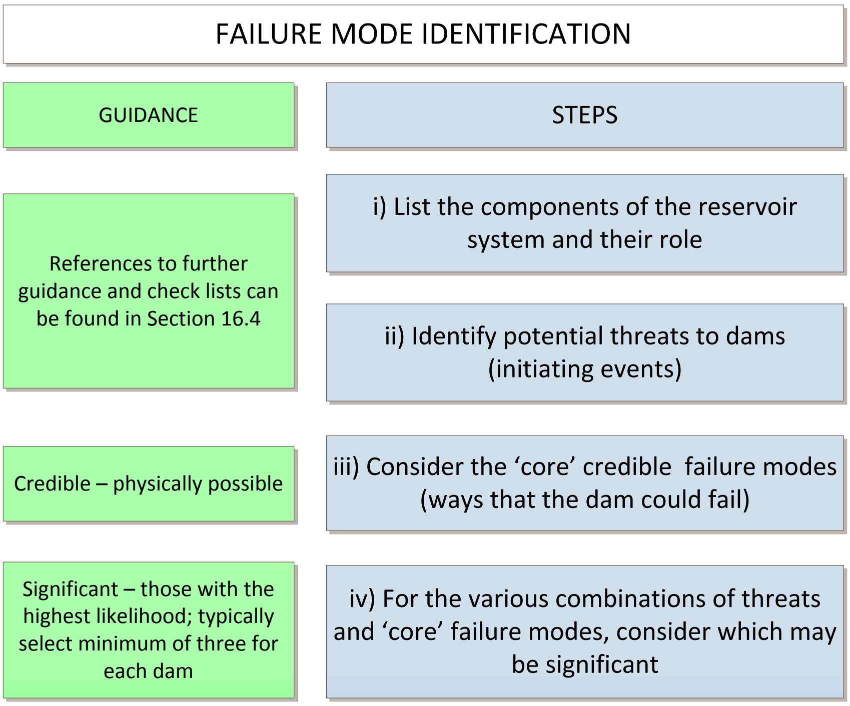
For **Tier 1** reliance is placed on available information on the dam and an initial list of **core** failure modes listed in Table 3.1. The FMI process for Tier 1 (|Figure 3.1) involves four steps:

- **Step i.** List all components of the reservoir system and their roles in preventing dam failure and interdependencies with other components.
- **Step ii.** Identify potential threats (initiating events) to the dam. List all significant external threats (initiating events or sources) that could initiate a failure mode. Refer to the list provided in Table 3.1 but do not be limited to this list. Similar checklists for these can also be found in Brown and Gosden (2004), Kennard et al.(1996a) and Johnson et al. (1999).
- **Step iii.** For each threat, based on the functional understanding of all components of the dam system, consider the potential ways in which the dam could fail (that is, core failure modes) that are credible (physically possible) as per the lists of core failure modes in Table 3.1. Do not be limited to this list. Additional information can be found in Environment Agency (2011a, 2011b). The description of the failure mode should differentiate between threat (initiation), failure mode (progression) and breach. Thus, for example, failure by sliding can occur due to several threats, namely elevated reservoir level in flood, earthquake or foundation

deterioration. Description of the failure mode therefore needs to include all of these elements of the overall failure process.

- **Step iv.** Classify all ‘core’ failure modes as credible or not credible, and as significant or not significant. Select the highest probable/most significant combinations of threats and failure modes to take forward in the risk assessment. Follow the process in the flow chart in Figure 3.2 to determine which failure modes are potentially credible and significant.

Figure 3.1 FMI process (for Tier 1)



It is recommended that:

- all the core threats are listed along with the reasons why they are not significant/credible
- a minimum of three failure modes is carried forward in the risk assessment (that is. a minimum of the three highest (least unlikely) failure modes,

If insufficient evidence is available to complete this step for some failure modes, then err on the side of caution by including them in the risk assessment, with the understanding that they may be excluded given further evidence.

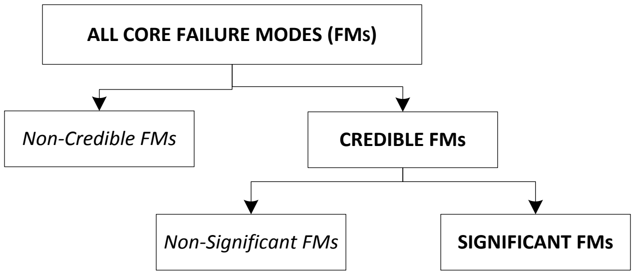
Where a potential failure mode is considered credible and significant but is not listed in Table 3.1, the user should either move to Tier 2 or 3, or develop a Tier 1 level of analysis to assess the likelihood of failure.

Table 3.1 Threats, failure modes and breach types (for Tier 1)

Initiation threat (root cause		Progression (failure mode)	Breach	Conditional probability of failure given load: reference in text	Failure scenario as referred to in Tier 2
	Table/section where further guidance is given	Those donated **** should be included in all risk assessments			
Embankment dams					
Floods	Rapid methods in FRS Appendix 1	Crest overtopping leading to scour	Embankment collapse	4.2.1	FL1
		Spillway chute overtopping		4.2.2	FL2
Wind generating waves	FRS chapter	Crest overtopping by waves leading to instability of downstream slope		4.2.3	Wi5
Various, for example, wave overtopping, intense rainfall		Saturation of downstream slope, leading to instability		Not included in Tier 1	FL1
Deterioration of body, or foundation of dam	Included in progression	Internal erosion – piping through the embankment ****		4.1	Db10
		Piping through foundation, or from embankment into foundation			Df10
		Piping along interface between structure and embankment	Local collapse along interface with structure		Ds10
		Leakage from pipe/through culvert leads to internal erosion along interface between structure and embankment			Ds1
Concrete dams and service reservoirs					
Flood (excessive inflow for service reservoir)	Likelihood of loading in Table 4.12	Crest overtopping leading to scour/erosion of earth fill providing support to concrete gravity structure – instability of gravity structure	Blocks slide/overturn	4.2.3	Fl1
		Sliding/overturning on lift joint			Fl6
		Sliding/overturning in foundation			Fl7
Earthquake		Sliding/overturning on lift joint		4.3.3	Eq6
		Sliding/overturning in foundation			Eq7
Deterioration of foundation of dam		Differential settlement initiated stability failure ****		4.3.4	Df7
Other deterioration		See Part 2			Various

FRS = *Floods and Reservoir Safety*, 3rd edition, ICE, 1996

Figure 3.2 Screening and classification of potential failure modes



1) Credible failure modes are physically possible even though they may be extremely unlikely to occur.
2) Significant failure modes: significance varies with the purpose of the risk assessment but in judging significance the cumulative effects of multiple failure modes should be considered.
3) Where uncertainty exists - err on the side of safety by:

- Initially classifying a failure mode as credible and significant; and
- Exploring the value of further investigation.

Box 3.1 Example output for Step 1a – failure mode identification

Failure Mode No.	Description of failure modes			Credible?	Justification	Significant ?	Justification
	Initiation (threat)	Progression (failure mode)	Breach				
Internal							
Ds1	Deterioration of pipework	Pipe burst under reservoir head, escaping through joints/cracks in culvert and eroding downstream shoulder fill	Embankment collapse	Yes	Requires joints/cracks in culvert before erosion can occur	No	Culvert likely to contain flow
Ds1	High water level during flood; Deterioration	Internal erosion along outside of by-wash	Collpase of spillway walls and erosion of slot through abutment	Yes	Cut-off wall extends under by-wash but upstream of road bridge; thus vulnerable area between cut-off and road bridge; base is concrete slab with open (previously bitumen filled) joints. Side walls mass concrete with rear of wall draiange; Side walls probably continuous with no joints.	Yes	However spillway is situated high up on abutment and would only lose limited depth of reservoir. Single estimate of consequences would overestimate the impact
External							
Fl1	Flood	Overtopping of crest and erosion of fill	Embankment collapse	Yes	Embankment downstream face could erode	Yes	Potential for blockage of part of bridge
Fl2	Flood	Overtopping of chute and erosion of fill	Embankment collapse	No	Chute is in cut through abutment. No access to embankment fill		

3.2 Step 1b – Identification of potential consequences

The area downstream of every reservoir is different. Topography, land use and occupancy all vary. Flooding of these areas will therefore result in different types and levels of impact. Level of impact will also depend upon the velocity and depth of inundation resulting from a dam failure.

The actual analysis of potential consequences of flooding is conducted in Step 2d. The purpose of looking at potential consequences at this point is to try to gain an appreciation of the magnitude of these consequences in order to:

- reconfirm that a Tier 1 (rather than Tier 2) analysis is appropriate
- appreciate the physical extent of potential inundation and hence data requirements for the assessment

The minimum consideration is to examine a 1:25,000 scale map to identify potential receptors downstream of the dam. It is often worth purchasing a map covering the downstream valley, particularly where a site visit is being made. Where internet access is available the search can be extended using similar online topographical maps (see Figure 3.4 for example).

First, consider what failure scenarios should be included, that is, ‘sunny day’ and/or ‘rainy day’ failure scenarios as defined below.

Scenario	**Definition of failure scenario**
Sunny day	Reservoir just full (that is no flood)
Rainy day	Reservoir at level of top of crest wall (if it can withstand overtopping); or dam crest level (where there is no wall, or the wall cannot withstand overtopping) (that is a flood condition)

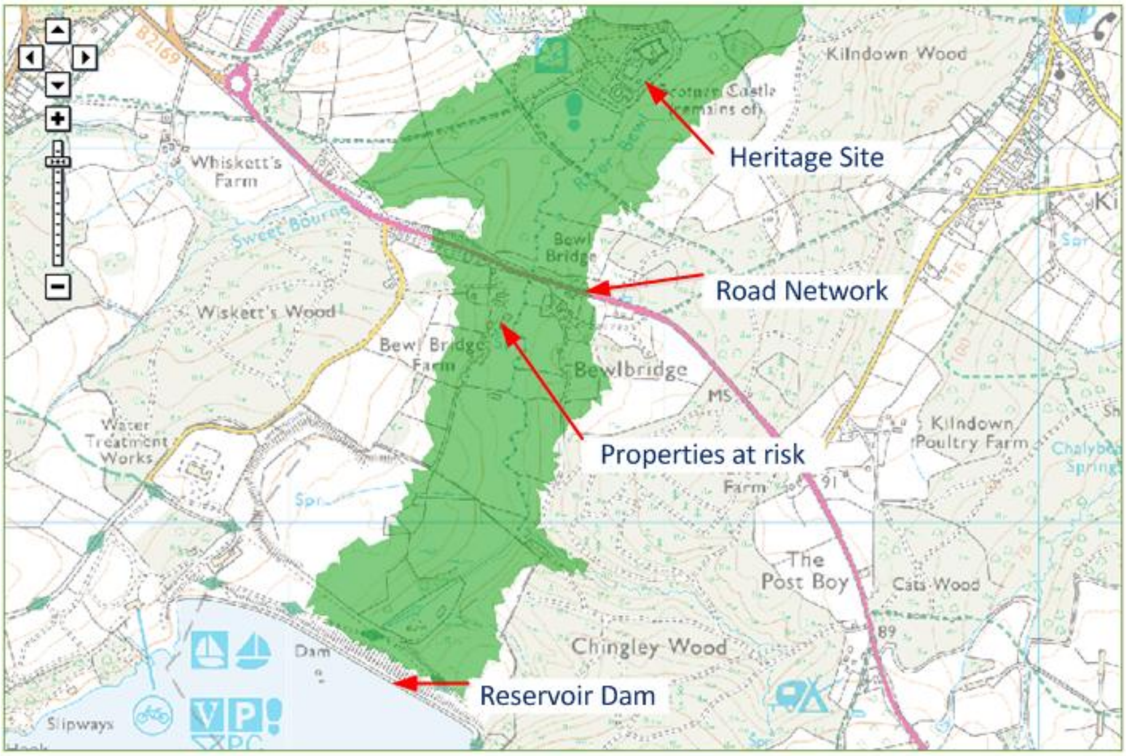
At Tier 1 it is normal to only consider the ‘rainy day’ scenario.

Next, estimate over what area inundation is going to occur.

Refer to a 1:25,000 scale map of the down-valley area (and/or fluvial flood maps downloaded from the Environment Agency website – see ‘What’s in my backyard’, reservoir flooding at www.environment-agency.gov.uk). Figure 3.3 shows an example reservoir inundation map taken from the website.

If dam break or flood maps are not available, undertake a quick assessment by assuming a water depth immediately downstream of the dam of half the dam height and follow contours down the valley to give an indicative inundation area. Look for likely downstream extents for the limit of inundation such as the valley opening to a larger river valley or floodplain.

Figure 3.3 Example reservoir inundation map



Then, for each of the headings in the table below consider which receptors are present and would be impacted by an uncontrolled release of water from the reservoir.

Type of receptor	Examples
Loss of life	Risk to life, health facility
Economic activity	Property damage, critical and transport infrastructure
The environment	Adverse impact to habitats and/or species
Cultural heritage	Damage to historic buildings, archaeological sites and so on

Identify the following:

- residential and commercial properties likely to be damaged/affected
- any transport infrastructure (motorways, A roads, railways) that could be affected
- any critical infrastructure – utilities assets (power, gas, water), communications assets, hospitals, police, fire and ambulance stations – that could be affected
- agricultural land that could be affected (optional where internet access is not available)
- designated environmental sites such as Sites of Special Scientific Interest (SSSIs), Special Areas of Conservation (SACs), Special Protection Areas (SPA's) and Local Nature Reserves (LNRs) that could be affected

- cultural heritage features (for example, historic buildings, monuments and archaeological sites) that could be affected

Box 3.2 Example output for Step 1b

Human Health	To be evaluated?	Yes
Identify centres of occupation where people could be at risk	Pennine way footpath, New East and Low Houses, Caravan Park Cotherstoen, Barnard Castle, Wycliffe Hall, Hedgeholme, Gainfo Piercebridge, Low Coinscliffe and the Edge of Darlington, along t banks of the Tees	

Economic Activity	To be evaluated?	Yes
Identify any transport infrastructure that could be affected	B6277, A67, B6274, A1(M) and A66(M) and a railway line, althou the motorways and the railway may be raised.	
Identify any critical infrastructure that could be affected	As above, including several sewage treatment works	

The Environment	To be evaluated?	Yes
Identify any agricultural land and environmental designations, and cultural hertiage sites that could be affected	Nature Reserves and part of a National Park	

Cultural Heritage	To be evaluated?	Yes
Cultural hertiage sites that could be affected	Remains of Egglestone Abbey (Premonstratensian), Roman fort, Roman remains of a bridge.	

Description of exposure scenario for which consequences are to be estimated	Rainy and sunny day failure, but inundation extents to be taken t Darlington to the A66(M). It is difficult to determine at this stage the extent of the inundation using the EA flood maps.

3.3 Step 1c – Review of outputs and scoping the risk analysis

Conduct a critical review of the outputs from Steps 1a and b, considering whether they can be carried forward, or whether any aspects of the assessments should be refined. (This could, for example, include the need for more accurate evidence, or moving onto a higher tier analysis.)

For Tier 1, the output would normally comprise that shown in Table 3.2.

Table 3.2 Summary of typical expected output from Tier 1 risk assessment

Failure mode		Consequence		Risk
Failure mode	**Loading condition**	**Scenario**	**Type**	
Credible and significant selected from list of core threats (Table 3.1)	Single 'dam critical' load	One (rainy day) only	Single value, selected to represent all of human health, economic activity, environment and cultural heritage	See Box 5.2 – single point on 5 × 6 risk matrix for each risk scenario.
Number of combinations provided in the output				
Combined total	1	1	1	1 *
* User can also choose to display subdivision of likelihood of failure, and/or consequences in output if required (see Box 6.1).				

Box 3.3 Management of uncertainty at Tier 1

Section 3.6 of Volume 1 describes the sources of uncertainty in any assessment. At Tier 1, single point estimates of the components of a risk assessment are made. Section 5.2.3 provides a suggested way to review and document the uncertainty in the analysis.

Consider the availability of data on the dam, and its quality. If there are not enough data available to complete Step 1a, a critical examination should be carried out of the data in relation to the failure modes for which the likelihood of failure is to be estimated, and a record made of what exists and its quality.

If there is not a currently valid condition assessment with sufficient information available about the condition of the dam to inform the risk assessment, consider performing a condition assessment. Refer to section 2.4.7.

4 Tier 1 – Steps 2a–b Likelihood of failure

Step 2	Risk Analysis	Step 2a	Likelihood of failure due to internal threats
		Step 2b	Likelihood of failure due to external threats

Step 2 builds on the output from the failure modes identification in Step 1 to decide which threats/failure modes are to be further considered. Step 1 should also have also ensured there are sufficient data to carry out an assessment of the likelihood of failure.

Note that the guide only provides guidance on evaluation of core Tier 1 threats/failure. If additional failure modes are required, reference should be made to the methodology in higher tiers and supporting information in Part 2.

NB: For ease of use this section presents Steps 2a and b together for embankment dams and then again for other types of reservoir dam. Embankment dams follow immediately below; for other dams types go to section 4.3.

Threats which could lead to dam failure can be subdivided into:

- external threats such as loading, impacts and desiccation
- internal threats such as cracking, internal erosion and suffusion

Internal threats are where the root cause of failure is within the body of the dam, or its foundation, caused for example by deterioration or ageing. Such causes may lead directly to failure under constant load, or may weaken the dam to such an extent that it fails rapidly when subject to a change in external load. These types of failure make up about half the causes of failure of dams in service (Brown and Tedd 2003).

4.1 Step 2a – Likelihood of failure due to internal threats for embankment dams

Internal conditions and processes within the structure of a dam can cause it to weaken or fail. The physical mechanisms controlling initiation and the rate of development of these internal threats are still not fully understood, and for internal erosion, are controlled by all of the three elements of material susceptibility, stress state and hydraulic load.

Ageing of the dam and how the dam has reacted to load in the past can affect the current stress state and the degree of consolidation of the dam. Thus it is necessary to consider both how the dam was built (which is the ‘intrinsic’ condition) and its current condition.

As the root cause of these effects is within the body of the dam it is difficult to measure what is happening inside the dam. Assessment therefore has to rely on external features and measurements (see Table 4.16 of surface indicators of internal erosion), any monitoring of parameters within the dam, and knowledge of the performance of similar dams.

First, consider likely significant combinations of threat/failure mode, typically comprising those shown in Table 4.1.

i. For each combination use the matrix in Table 4.3 to determine the likelihood of failure of embankment dams based on intrinsic and current condition. Further guidance on selecting the scores is given in Tables 4.17 and 4.18.

NB: A qualitative measure of the range of probability (that is, low, moderate, high and so on) is provided in section 15.2.2.

Table 4.1 Core 'Tier 1' Internal threats to embankment dams

Threat	Progression (failure mode)
Crack in erodible watertight element, for example, due to: a) differential settlement, or desiccation of core b) hydraulic separation between structure and adjacent fill	Concentrated erosion along sides of crack
Internally unstable soil forming dam/ foundation	Suffusion (loss of fines from within matrix)
Foundation includes fine soil overlying coarse soil	Contact erosion along contact
High hydraulic gradients in erodible soils	Backward erosion (piping)
Notes [1] At least one of the above must be included in all risk assessments, as internal erosion is one of the main causes of dam failure.	

The frequency of surveillance of a dam can influence the likelihood of failure. If the surveillance interval is too long compared with the potential time from initiation of the failure process to breach then it is unlikely that the breach could be prevented.

Consideration should also be given to the type of soil that forms the core of the dam and the length of time would potentially take to breach once the failure process starts.

Table 4.2 gives guidance on what would be considered rare and infrequent surveillance frequencies for different common core soils and foundations in embankment dams. Compare this with the actual surveillance frequency for the dam and use judgement to adjust the condition score for the Table in 4.3 accordingly.

Table 4.2 Guidance on frequency of surveillance for use in assessing likelihood of failure due to internal threats

Speed of failure		Soil forming core/dam foundation (take worst case)	Frequency of surveillance	
Descriptor	Time from failure to initiation		A	B
Fast	<7 days	Non-cohesive soils, dispersive soils	3 days	2 days
Medium	8–90 days	Low plasticity clays	15 days	7 days
Slow	90 days	High plasticity clays	30 days	15 days
Notes	Assume gravity dams on rock foundations have 'medium' speed of failure.			

An example of the outputs expected from Step 2a are shown in Box 4.1.

Box 4.1 Example output for Step 2a – likelihood of failure of embankment dams due to internal threats

Internal Threat Failure Mode 1		
Piping embankment	Threat	Deterioration of body, or foundation of dam
	Failure mode	Internal erosion
	Score	Remarks
Determine intrinsic condition score, and state which factors led to this value	2	1960's design with filtered core & foundation treatment; no signs of problems
Determine current condition score, and state what factors led to this value	2	Low plasticity clay core; normally visited weekly unless weather conditions prevent. Table 3.5 'infrequent'
Complete matrix (Table 4.3) to determine likelihood of failure	Moderate	
Internal Threat Failure Mode 2		
Piping founation	Threat	Deterioration of body, or foundation of dam
	Failure mode	Internal erosion
	Score	Remarks
Determine intrinsic condition score, and state which factors led to this value	2	1960s' design with filtered core & foundation treatment; no signs of problems
Determine current condition score, and state what factors led to this value	2	
Complete matrix (Table 4.1.3) to determine likelihood of failure	Moderate	

Table 4.3 Matrix for assessment of likelihood of failure due to internal threats

Intrinsic condition		Current condition (for example, performance features (seepage, deformation), frequency of surveillance (see Table 4.2), operation of reservoir)				
Score	Vulnerability to failure due to internal threats	1	2	3	4	5
		No signs of adverse behaviour	A few symptoms, or surveillance less often than frequency 'B'	Some symptoms, or surveillance less often than frequency 'A'	Symptoms of structural issues (for high consequence dams may lead to works)	Symptoms of serious structural problem, leading to emergency drawdown
5	**Direct evidence**	Low	Moderate	High	Very high	Extreme
4	**Evidence suggests it is plausible, and is weighed more heavily towards likely than unlikely**	Low	Moderate	High	Very high	Extreme
3	**As above but key evidence is weighted more heavily toward unlikely**	Low	Low	Moderate	High	Extreme
2	**The probability cannot be ruled out, but there is no compelling evidence to suggest that a flaw exists**	Very low	Low	Moderate	High	Very high
1	**Several features must occur concurrently to trigger failure – most are very unlikely**	Very low	Low	Moderate	High	Very high
Notes	Guidance on frequency of surveillance in given in Table 4.2.					

4.2 Step 2b – Likelihood of failure due to external threats for embankment dams

The following sections provide guidance on the likelihood of failure arising from the main external threats to embankment dams including crest overtopping, chute overtopping and slope instability.

4.2.1 Likelihood of failure due to crest overtopping – embankment dams

Overtopping can occur when water levels in a reservoir are near the crest height of an embankment dam, such as following heavy rains or storms. Such overtopping can cause significant erosion and damage to the downstream face of the dam, which can in turn lead to dam failure.

To prevent dams overtopping, spillway weirs are normally installed to draw off excess water from the reservoir to lower the water level. The likelihood of crest overtopping of an embankment dam under flood conditions can thus be related primarily to the capacity of the spillway relative to the flood load. The capacity of the spillway to pass the flood is also affected by its susceptibility to blockage by debris.

In many cases an estimate of the capacity of the spillway to pass the probable maximum flood (PMF) will already be available from previous inspections or analyses. For a Tier 1 analysis this will be appropriate. Where an estimate is not readily available, then a quick estimate may be made by using the method given in Appendix 1 of *Floods & Reservoir Safety* (ICE 1996).

The capacity of the spillway weir can be calculated using Figure 4.1. This requires the following parameters:

a) Freeboard (the height from the spillway crest to the lowest point on crest of the dam). Use this to read off from Figure 4.1 the spillway capacity per metre width – assume flood level at dam crest (that is, no allowance for wave freeboard at point of imminent failure due to crest overtopping).

   For smaller reservoirs, the coefficient of 1.5 applies.

   Where a weir has a flat top (broad crested), use a coefficient of 1.5.

   Where the width of the weir top is less than a third of the depth of flow over the weir at flood, use a coefficient of 1.7.

   For further guidance on spillway weir coefficients, refer to BS ISO 3846: 2008 *Hydrometry. Open channel flow measurement using rectangular broad-crested weirs* (BSI 2008).

b) Width of the spillway. Multiply the number in ‘a’ by this to obtain the total spillway capacity.

Use Table 4.4 to determine a failure likelihood based on the spillway weir capacity in relation to the PMF at the reservoir (that is, the ratio of the overflow capacity to the PMF).

NB: The spillway ratios provided (that is, 0.2, 0.3, 0.5 and so on) relate to the design capacities suggested in Table 1 of *Floods and Reservoir Safety* (ICE 1996).

When assessing the likelihood of failure, consideration should also be given to the likelihood of debris blockage during a flood event. Where there is a significant chance of blockage, the effective spillway capacity is reduced; suggested values are given in Table 4.5.

Figure 4.1 Flow per metre width of spillway weir

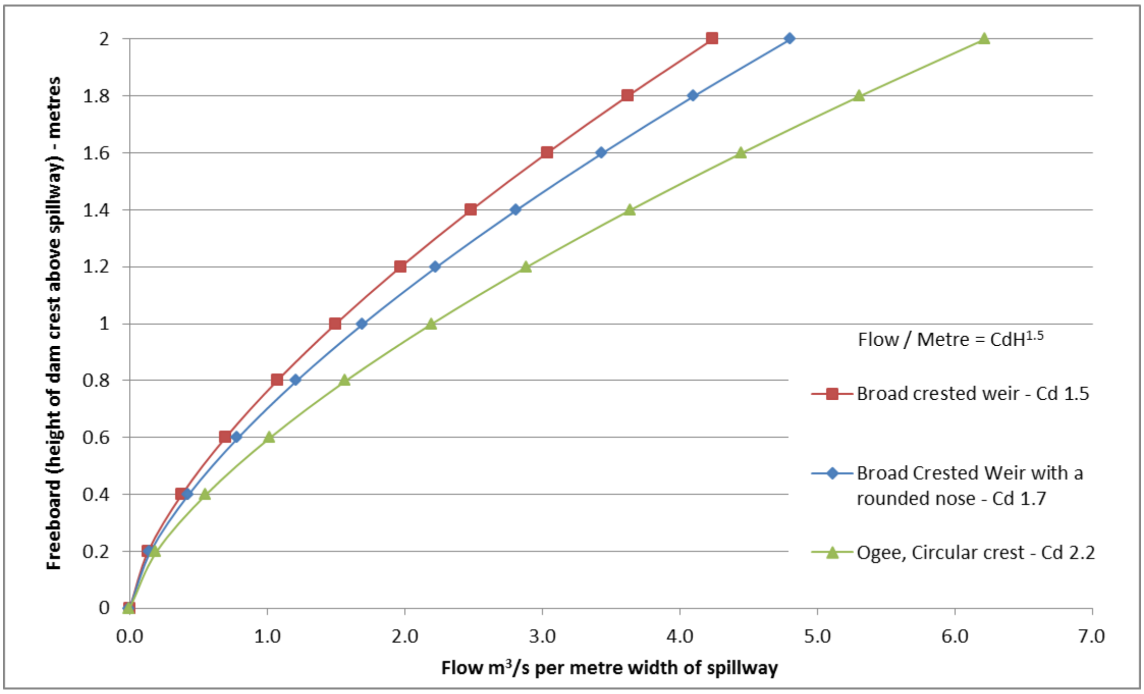


Table 4.4 Indicative likelihood of embankment dam failure due to crest overtopping

Weir capacity (allowing for blockage)/routed PMF		Likelihood of failure due to lack of spillway	Likelihood score
	<0.2	<100	Extreme
0.2	0.3	100-1,000	Very high
0.3	0.5	1,000–10,000	High
0.5	0.8	10,000–100,000	Moderate
0.8	1.1	100,000–1,000,000	Low
>1.1		>1,000,000	Very low

Table 4.5 Preliminary values of blockage of spillways due to floating debris

Size of opening (lesser/minimum dimension)	% blockage of spillway weir/chute, where trees are present, either around the reservoir or on any incoming watercourse, within 1km of reservoir
>10m	Nil
5–10m	10
2–5m	25
<2m	50
Notes	Where there are multiple openings (pipes or arches), adjust as appropriate (that is, either use combined width of all openings, or for larger openings assume only opening is blocked, so that % reduced by proportion of arches blocked to total number of arches).

Box 4.2 Example output for crest overtopping calculation

Floods; leading to crest overtopping			
	Value / Score	Units	Remarks
Calculate PMF	16	m^3/s	Routed winter PMF (summer peak inflow is higher but outflow is lower); peak inflow is $28m^3/s$
Weir capacity	14	m^3/s	Take to 300mm above embankment crest, only as permeable wave wall unlikely to stand significant stillwater loading; flat ogee crest take coefficient of 2. Capacity at crest level is $7m^3/s$. Capacity to wave wall crest $40m^3/s$
Risk of blockages from debris	Low		No trees in catchment
Potential % reduction of blockage to overflow weir	0%	%	
Revised capacity of overflow weir	14	m^3/s	
Overall likelihood of failure (weir capacity/routed PMF)	0.88		Revised capacity/PMF = 0.88
Using Table 4.2.1	LOW		

4.2.2 Likelihood of failure due to spillway chute overtopping – embankment dams

Spillway chutes channel water from the spillway weir safely to the downstream watercourse. Overflow from these chutes can erode the surface of an embankment dam.

NB: If overflow is unlikely to erode or affect the dam then there is no need to continue with the analysis of spillway chute failure.

An approximate estimate of the likelihood of flow exceeding the capacity of a spillway chute, and eroding the adjacent embankment may be estimated as follows:

a) Select the point (or points) where flow is likely to be lowest. You will need the following dimensions for each point:

- bed width
- bed slope
- height of wall

b) From Figure 4.2 read off the spillway capacity per metre width. It is suggested that 0.15m is added to the depth to obtain the depth of overtopping that is likely to cause significant erosion of the adjacent bank

c) Width of the chute. Multiply the number in ‘b’ by this to obtain the dam critical flow at that point in the chute.

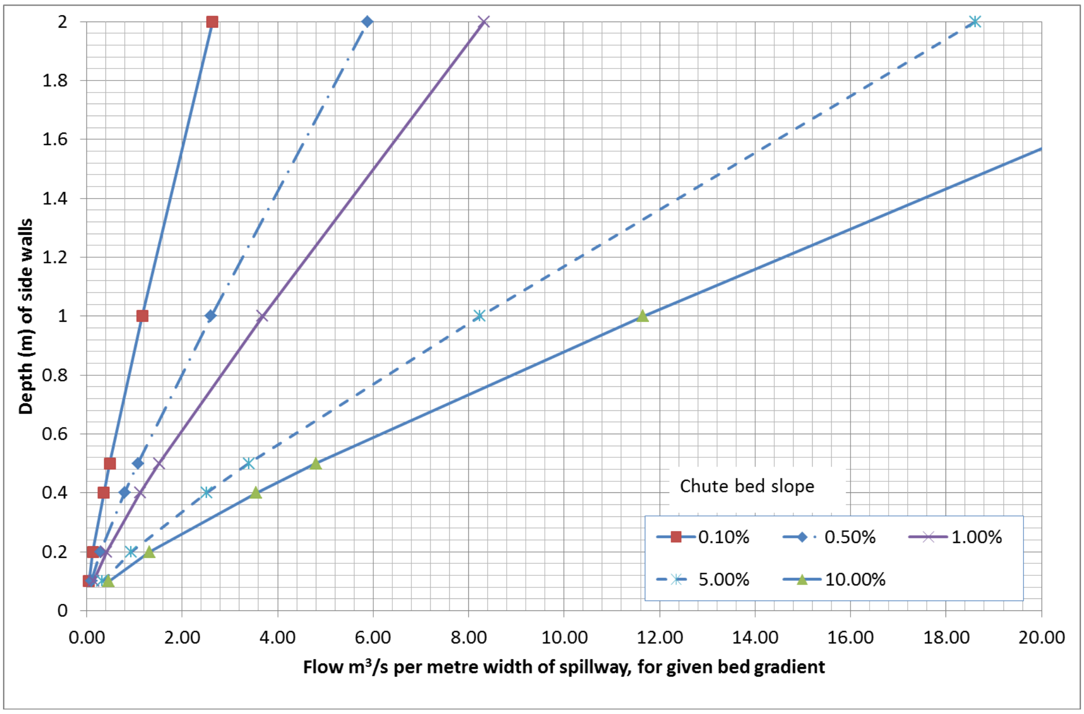
d) If there are any bends in the chute then water is likely to escape the chute more easily. In this case halve the dam critical flow. NB: If this results in a high risk then use Tier 2 analysis to refine the estimate.

e) This is the ‘adjusted chute critical flow’ at that point. Repeat this for the points in the chute that may be critical (pinch points) and use the lowest estimate as the overall chute critical flow.

f) Plot a graph of flow vs. annual likelihood, using your knowledge of PMF (which should be taken as annual chance of 1 in 400,000) and read off the annual chance of the flow corresponding to the ‘adjusted chute critical flow’.

g) Use Table 4.2.4 to read off the likelihood.

Figure 4.2 Flow per metre width of spillway chute

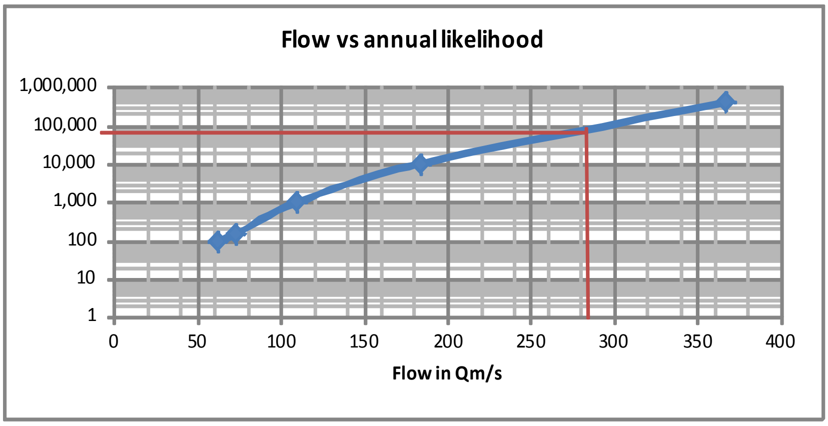


Record the outputs of the external threats assessment and the likelihood of failure scores in a similar way to those shown in Box 4.3.

Box 4.3 Example output for spillway chute overtopping calculation

Chute Overtopping	Value / Score	Units	Remarks
Select point(s) where flow is likely to be the lowest			One point estimated for example
Bed width	20	m	
Chute bed slope	5%	%	
Height of wall	1.5	m	
Depth to side of wall (+0.15m)	1.65	m	Height of wall + 0.15
Spillway capacity per m	13.5	m^3/s	Taken from figure 4.2
Dam critical flow at point(s) (spillway capacity per m * bed width)	270	m^3/s	13.5*20 = 270
From graph plotted - annual chance for chute	1 in 65,000 years		
Table 4.4 - Likelihood score	Moderate		

Year	Q - Peak inflow
400,000	366.77
10,000	183.39
1000	110.03
150	73.35
100	62.35



4.2.3 Likelihood of failure due to instability – embankment dams

The slopes of embankment dams can destabilise with loads that exceed their capacity to resist them, or due to external and/or internal erosion mechanisms. Thus the likelihood of failure needs to consider the likelihood of a change which destabilises the slope (load), then the conditional probability of slope failure given that load, and finally the conditional probability of release of the reservoir, given the slope failure.

Calculating these forces (for example, extreme rainfall, overtopping of the dam crest by waves) and responses (leaks through the body of the dam) can be a lengthy and complicated process. For simplicity and rapid assessment at Tier 1, this assessment compares the downstream slope of the dam against a typical slope that would be expected for a modern dam. Indicative slope angles used in ‘modern design’ for earthfill dams are given in Table 4.6.

Refer to the guidance provided in Table 4.8. It is important to differentiate between a slope failure of the downstream face but does not extend across a wide crest, and a slope failure which removes most of the crest and presents an imminent risk of release of the reservoir. Thus the width of the dam crest as contributing to the likelihood of release of the reservoir is included in the form of a ratio with the dam height.

Table 4.6 Indicative 'modern design' slopes for earthfill dams

Soil type	Downstream slope on good foundation	Source
Sand, gravel	2.5H:1V	Section 9.2.3 of CIRIA Report 161 (Kennard 1996a)
Low plasticity clays	3.0H:1V	
High plasticity clays	4H:1V	Figure 10 of Vaughan et al. (1979). For more detailed assessment where slope angle is related to geological origin of the construction material reference can be made to Table 4 of Parsons and Perry (1985).
Notes	[1] Where pre-existing shear surfaces are present at the ground surface (for example, due to periglacial action), then much flatter slopes would be required. For example, the redesign of Carsington dam adopted flatter slopes (Johnston et al. 1999) and overall slopes of around 10H:1V have been required on some dams to ensure foundation stability.	

Table 4.7 Slope instability likelihood categories

Comparative stability (in relation to hazard of release of reservoir)	Criteria to assess stability (measured relative to modern slope design, as defined in Table 4.5)	Likelihood score for crest width C as ratio to dam height H		
		C/H < 0.5	0.5 < C/H < 1.5	C/H > 1.5
Very unstable	Slope which is >25% steeper	Extreme	Very high	High
Potentially unstable	Slope which is up to 25% steeper	Very high	High	Moderate
Borderline	Slope which is up to 15% steeper	High	Moderate	Low
Modern dam	Slope designed to modern understanding of geotechnics	Moderate	Low	Low
Very stable	Slope with slope angle 15% or more shallower	Low	Low	Very low

Table 4.8 Likelihood of release of reservoir given slope instability

Criteria		Reduction in likelihood of reservoir release compared to slope failure (from Table 4.6) (number of classes)	Comment
Base case: 1m freeboard, 3m wide crest, slip would be noticed within three days		3	For example, Very high becomes Low
Factors varying above (user judgment, or interpolate. Extrapolate below to suit individual dam)			
Less likely		**More likely**	
Crest 6m wide	0.3	Crest 1m wide	-0.5
Freeboard 2m		Freeboard 0.2m	-0.5
		Vertical wall along upstream face, so water line on upstream side of crest	-0.2
		Reduced surveillance, say one month	0.2
Sum all factors to obtain likelihood of reservoir release due to slope instability		For example: 3.0 + 0.2 (5m wide crest) 0.5 (0.2m freeboard) -0.2 (vertical wall) = 2.5. Take as 2 to be conservative. So Very high becomes Moderate.	

4.2.4 Likelihood of failure due to other external threats – embankment dams

Other external threats include items such as subsidence, terrorist activity and plane crash impact. These threats would not normally be considered until a Tier 3 analysis.

Box 4.4 Example output for instability of embankment dams

Slope instability	Value/Score	Units	Remarks
Slope angle	2.5		
Crest width	11	m	36 feet (11m); dam height 12m; C/H 0.9
Modern design standard slope angle (Table 4.6)	3.0	Modern slope angle	Assume low plasticity clay Using Table 4.6 – modern design standard slope = 3.0H:1V based on Kennard *et.al* (1996a)
Difference to modern slope design	17%	%	1-(slope angle/modern design)
Stability output (Table 4.7)	Potentially unstable		Slope is up to 25% steeper
Likelihood	High		

Subsidence can be caused by a number of initiators including:

- mining
- dissolution of limestone in karst terrains
- gas and oil extraction
- geological faulting
- isostatic (crust) movement
- seasonal effects (soil moisture content)

If any of these threats are considered significant then a Tier 3 risk analysis is recommended.

4.3 Step 2 – Likelihood of failure for dams other than embankments

4.3.1 Likelihood of failure due to internal threats

Concrete dams, masonry dams and service reservoirs are also subject to internal threats to the stability and integrity of their structure. For concrete gravity structures internal threats which could lead to failure are summarised in Table 4.9.

Table 4.9 Internal threats to concrete gravity structures (concrete/masonry and service reservoirs)

<table>
<tr><th rowspan="2">Threat/FM (code in Table 3.1)</th><th colspan="2">Concrete dams</th><th colspan="2">Service reservoirs</th></tr>
<tr><th>Threat</th><th>Progression (FM)</th><th>Threat</th><th>Progression (FM)</th></tr>
<tr><td>Df10</td><td>Deterioration of dam foundation</td><td>Sliding</td><td>Deterioration of dam foundation</td><td>Sliding</td></tr>
<tr><td>Db6/Db7</td><td>Blockage of internal/foundation drains</td><td>Sliding</td><td colspan="2">Service reservoirs generally do not have internal/foundation drains, so not a vulnerability</td></tr>
<tr><td>Li7/Li11</td><td colspan="2">Most concrete dams do not have reservoir lining so generally not a credible threat</td><td>Leakage from Service reservoir</td><td>Increase in uplift pressure/sliding
Internal erosion of foundation</td></tr>
<tr><td>Ds1</td><td colspan="2">Pipe burst generally not a significant threat to concrete dams</td><td>Fracture/leak of pipes under pressure passing through external fill, where the perimeter wall relies on the fill for structural support</td><td>Saturation/failure of perimeter bank, leading to low of support of perimeter wall</td></tr>
</table>

Use the matrix in Table 4.3 to determine the likelihood of failure of concrete and service reservoir dams based on intrinsic and current condition. Further guidance on selecting the scores is given in Tables 4.19 and 4.20.

Box 4.5 Example output for likelihood of failure for concrete/masonry dams and service reservoirs due to internal threats

Failure Mode No. (Table 3.1)	Initiation (threat)	Progression (failure mode)	Intrinsic condition		Current condition		Complete matrix t determine likelihood of failu
			Score (Table 4.19)	Factors which led to this value	Score (Table 4.20)	Factors which led to this value	
Df7	Deterioration of dam foundation	Differential settlement initiates failure	2	Low - The probability cannot be ruled out, but there is no compelling evidence to suggest that a flaw exists	1	No signs of adverse behaviour	Low

4.3.2 Likelihood of failure due to external threats

The external threats for concrete structures in a Tier 1 assessment are listed in Table 4.10. It should be emphasised that in order to provide a qualitative method several significant simplifications have been made, using a precautionary approach. Where the likelihood of failure is a significant issue then the user should move to a Tier 2 analysis.

Table 4.10 Core external threats for concrete gravity structures

Failure scenario (see Table 3.1)	Failure mode	
	Concrete dams	**Service reservoirs**
FL1	Elevated water level causes instability on construction 'lift 'joint	
FL7	Elevated water level causes instability in foundation	
FL1	Concrete dams generally not vulnerable to scour of downstream fill	Overtopping/ scour of supporting fill
Eq6/Eq7	Earthquake causes Instability on construction 'lift 'joint/foundation	

Likelihood of failure due to floods

Use the matrix in Table 4.11 to determine the likelihood of failure due to external threats.

Table 4.11 Matrix for assessment of likelihood of stability failure due to flood

Likelihood of critical flood occurring		Vulnerability to stability failure given external load occurred (read off Table 4.12, corrected for any external fill using Table 4.13)			
Load likelihood	**Flood dam Consequence Class (Note 1)**	**Very unlikely**	**Unlikely**	**Neutral**	**Likely**
L1	–	Moderate	High	Very high	Extreme
L2	D	Low	Moderate	High	Very high
L3	C	Low	Low	Moderate	High
L4	B	Low	Low	Low	Moderate
L5	A	Low	Low	Low	Low
Notes	[1] Where the spillway has been designed to pass floods in accordance with *Floods and Reservoir Safety* (ICE 1996), it is suggested that a load likelihood is taken as corresponding to the consequence category shown above. For example, a reservoir with a Category B spillway would be classed as Likelihood L4. [2] Where the dam has not previously been assessed for safety under floods it is suggested that Likelihood L2 (Class D) is used, except where there are signs of overtopping of the dam when Likelihood L1 should be used. [3] Where trees are present and a spillway is vulnerable to blockage then increase				

	likelihood, for example, 5 to 4 as appropriate (using Table 4.5 to guide whether the likelihood is increased by one increment). [4] Covered service reservoirs can fail due to excessive inflow leading to spillage and water potentially rising to the roof. This will depend largely on whether there are any alarms, whether they are working and if any notice is taken of them, as well as any overflow capacity and the maximum rate of inflow. Where records show that the service reservoir has overflowed then a Likelihood L1 should be applied. Where there are alarms that are working and there are no records of overflow then it is suggested that Likelihood L2 is applied.

Box 4.6 Example output for likelihood of failure due to flood loading of a concrete dam

Flood loading concrete structures	Value	Units	Remarks
Body of structure (lift joint)			
B/H with flood at dam crest	0.53		9.6m wide at base divided by dam height of 18.14m = 0.53
Likelihood of failure using Table 4.12	Neutral		Within 'neutral' range
Likelihood of critical flood using Table 4.11	Low		Category A dam. Refer to line L5
Potential % reduction of blocked overflow weir	0%	%	Spillway >3m wide so no consideration of reduction required
Revised load likelihood	Low		Unchanged, no reduction
Overall likelihood of failure	Low		
Foundation			
B/H with flood at dam crest	0.53		9.6m wide at base divided by dam height of 18.14m = 0.53
Likelihood of failure using Table 4.12	Likely		Within 'likely' range
Likelihood of critical flood using Table 4.11	Low		Category A dam. Refer to line L5
Potential % reduction of blocked overflow weir	0%	%	Spillway >3m wide so no consideration of reduction
Revised load likelihood	Low		Unchanged, no reduction
Overall likelihood of failure	Low		

Table 4.12 Indicative likelihood of stability failure of concrete structure due to imposed water load

Location of failure	B/H (base width/depth of water, both measured at level of potential failure			
	Likely	Neutral	Unlikely	Very unlikely
Body of structure (lift joint)	<0.52	0.52–0.76	0.76–1.05	>1.05
Foundation	<0.85	0.85–1.0		

Table 4.13 Indicative likelihood of failure of earthfill providing support to concrete gravity structure

Failure mechanism	Adjustment to probability of failure of dam wall, for effect of downstream structural fill to full height of dam, with slope		
	≤1.5H:1V	1.5 to 2.5	≥ 2.5H:1V
Scour due overtopping	No adjustment	Reduce by one increment	Reduce by two increments
Instability under static reservoir load	No adjustment	Reduce by one increment	Reduce by two increments

Box 4.7 Example output for failure of fill material to provide structural support to a concrete structure

Fill material support to concrete structure	Value	Units	Remarks
Likelihood of failure using output from Table 4.11 (flood loading) in Box 4.6	Low		Output from flood loading
Fill material downstream slope	2.2:1		Downstream slope 2.2H:1V
Likelihood of failure using Table 4.13	Reduce by 1 increment		Within '1.5-2.5H:1V' range
Revised load likelihood	Low		Reduce by 1 increment
Overall likelihood of failure	Low		

Likelihood of failure due to earthquake

To assess the likelihood of stability failure due to earthquake enter the geometry of the gravity structure into Table 4.14, and then read down to the bottom of the column to read off the probability.

Table 4.14 Matrix for assessment of likelihood of stability failure of gravity dam due to earthquake

Vulnerability to stability failure given earthquake	Very Likely	Likely	Neutral	Unlikely
B/H (base width/ depth of water, both measured at level of potential failure)	**<0.38**	**0.38–0.62**	**0.62–1.2**	**>1.2**
Likelihood of failure due to earthquake	Moderate	Low	Very low	Very low

4.3.3 Overall likelihood of failure

The overall probability of failure is assessed as follows.

a) Summarise the likelihood of failure of all the credible and significant failure modes (see Table 4.15 for example).

b) The overall likelihood is determined as follows.

   i. Where there is a single maximum value use this.

   ii. Where there are two or more with the same maximum value, then use the next increment up for example if there are two (or three) 'H', then the overall likelihood is 'VH'.

Table 4.15 Example table of total likelihood of failure

Threat	Progression (failure mode)	Likelihood of failure
Floods	Crest overtopping	Low
	Chute overtopping	High
Internal threats	Body of dam	Moderate
	Foundation	Moderate
	Interface between structure and embankment	High
Overall likelihood of failure (use in risk matrix in Box 5.3.1)		Very high

4.4 Supporting tables for Tier 1 likelihood of failure

Table 4.16 Definitions and surface indicators of different types of internal erosion of embankment dams (adapted from ICOLD 2012)

Type of internal erosion	Definition	Time to failure; remarks	Locations	Common surface indicators	
				Dam crest	Downstream face/ toe
Concentrated erosion	In soils which are capable of sustaining an open crack. Erosion occurs along the sides of the crack where the shear stress (velocity) exceeds the critical value. NB: At low flows there may be leakage with no erosion.	The rate of erosion is dependent on the erosion resistance of the clay core, and may be limited by the permeability of the upstream and/or downstream shoulders. Where cracks exist in the dam crest (for example, desiccation, differential settlement), the critical failure mode may be concentrated erosion during flood conditions.	Wherever a crack can occur.	Sinkholes or local depressions: a) over the core where core material continually collapses b) where the core material can sustain an open arch then the hole may migrate upstream causing sinkholes or settlement in the non-cohesive material immediately upstream of the core (which cannot sustain an arch). In extreme situations there may be whirlpools	Seepage, suspended fines commencing at critical flow rate. Seepage may be concentrated in homogenous dams, or diffuse in zoned dams where the crack is in the core and the downstream shoulder does not retain fines.
Backward erosion (piping)	Erosion starts at the exit point; a continuous passage is developed by backward erosion when the seepage gradient exceeds the ‘flotation gradient’ of the soil.	Can be fast with little warning. Failure is often associated with first filling, or an increase in seepage gradient (for example, under flood conditions).	Where a pipe can be sustained	Generally no significant settlement, as for the pipe to be sustained the overlying materials form an arch. Some settlement may occur where the pipe forms partway through the dam, collapses, and reforms	Seepage with fines. In some instances, particularly flood defence embankments, small sand boils have been observed.
Contact erosion	Erosion at the horizontal boundary of a fine soil overlying a coarse soil, where the fine soil is washed into the coarse soil due to horizontal flow.	Little information	Where a fine soil overlies a coarse soil, at the contact for example flood embankments where a fine alluvial soil overlies a clean gravel	There may be some settlement, but this is only likely to be detectable when significant erosion has occurred.	Seepage with fines
Suffusion	Mass erosion in soils which are internally unstable. Fines transported by seepage flow between the larger sizes of soil.	Normally leads to an increasing quantity of seepage as fines erode, but is unlikely to lead to rapid failure.	At the elevation where the seepage velocities are highest in relation to the soil properties at that elevation	In theory there should no settlement, as it is loose fines from within the soil skeleton being eroded, with the soil skeleton remaining unaffected.	Seepage increasing with time until all fines are eroded or the increasing seepage triggers a slope instability or other change in conditions.

Table 4.17 Supplementary guidance on assigning intrinsic condition score for embankment dams (Tier 1)

Intrinsic condition	Extent to which feature means dam is vulnerable to failure, that is, criticality in failure modes analysis	
	Embankment	**Foundation**
	Embankment shoulder does not act as a filter to core	Erodible or compressible foundation
5 – Body of dam/foundation vulnerable to failure	Hydraulic gradient across core > 5	No foundation treatment such as slush grout/dental concrete on open jointed hard rock foundation
4	Erodible core material (silt or dispersive)	
3	Downstream slope steeper than 2H:1V Abutment slopes > 1V:1H or steps > 0.1H No filtered drainage in downstream shoulder	No foundation cut-off
2	Core material low plasticity clay	
1 – Design/construction inherently resistant to failure	Filtered core	On in situ rock, which is low permeability/been adequately treated to reduce risk of internal erosion
Notes:	Selection of score is judgement by user. Either take highest score (worst case) across both columns as giving condition (not average or minimum), or where several vulnerable features combine to give higher score. Where unsure (for example, no drawings) then do not score zero, but score most likely condition (for example based on typical construction practice at time the dam was built or upgraded).	

Table 4.18 Supplementary guidance on assigning current condition score for embankment dams (Tier 1)

'Current condition scoring' system for probability of failure due to internal threats

Current condition	Extent to which feature is symptomatic of performance and thus likelihood of failure			
	Seepage Quantity, fines	Deformation	Surveillance Monitoring	Reservoir operation/ability to lower reservoir
5 – Emergency drawdown	Fines being carried in seepage	Sinkhole > 1m deep		
4 – Some concern	Leaks a lot, could be carrying fines	Settlement increasing with time (>20% increase in a year		
	Seepage increasing with time (>20% increase in a year)			
	Seepage > 10 times Seepage Index given in Charles et. al. (1986, p. 7)	Settlement rate >3 times expected from Johnston et. al. (1999, p.16)		
3	Seepage quantity varies with reservoir level	• New local settlement > 0.1m deep • Persistent crack which relate to credible failure mode of length > dam height	Surveillance <2 per week in dams which are vulnerable to rapid failure (Note 2) No surveillance (dam not vulnerable to rapid failure)	
2	Extensive decaying tree roots in the vicinity of the crest/watertight element	Deep animal burrows where there could be risk to watertight element	• No instruments at dam, or readings not evaluated within one week of reading	• Never been filled for example flood detention reservoir
			• Poor ability to inspect (that is, large leak would not be detected	• No fixed bottom outlet/means of lowering reservoir in an emergency
				Annual refill is rapid (>10% of dam height/week)
				Rate of lowering with fixed bottom outlet < Hinks formula
1 - No signs of adverse behaviour	Seepage < Index given in Charles et.al (1986, p. 7)	No differential settlement. Total settlement < Johnston et.al (1999, p. 6).		Effective bottom outlet which can lower > Hinks formula

Notes: [1] Selection of score is judgement by user. Take highest score (worst case) across all columns as giving condition (not average or minimum). Where unsure (for example if no settlement or seepage monitoring) then do not score zero but score most likely condition.
[2] Dams which include one or more of the following are vulnerable to rapid failure – non cohesive core, sandy foundation, outlet pipe in cut and cover trench with no sand collar filter

Table 4.19 Supplementary guidance on intrinsic condition score for concrete gravity structure (Tier 1)

<table>
<tr><th rowspan="3">Intrinsic condition</th><th colspan="3">Extent to which feature means dam is vulnerable to failure that is criticality in failure modes analysis</th></tr>
<tr><th colspan="2">Body of structure</th><th rowspan="2">Foundation</th></tr>
<tr><th>Concrete/masonry gravity dam; service reservoir with mass concrete perimeter wall</th><th>Service reservoir constructed of reinforced concrete</th></tr>
<tr><td rowspan="2">5 – Body of dam/ foundation vulnerable to failure</td><td rowspan="2">Poor quality mortar in masonry sections</td><td rowspan="2">Thin section with no attention to lift joints</td><td>Erodible or compressible foundation</td></tr>
<tr><td>No foundation treatment such as slush grout/ dental concrete on open jointed hard rock foundation</td></tr>
<tr><td>4</td><td></td><td></td><td></td></tr>
<tr><td rowspan="2">3</td><td>Vertical joints with no waterstops/ shear keys</td><td>Frequent expansion joints with no shear connections</td><td rowspan="2">Dams – no grout curtain</td></tr>
<tr><td>No attention to bond on lift joints</td><td></td></tr>
<tr><td>2</td><td></td><td></td><td></td></tr>
<tr><td>1 – Design/ construction inherently resistant to failure</td><td>Modern well compacted concrete with positive design and construction measures to ensure high bond across lift joints and water stops in vertical joints</td><td>Reinforced concrete wall structurally connected to base and roof, with full continuity of steel across joints</td><td>On in situ rock, which is low permeability/been adequately treated to reduce risk of internal erosion</td></tr>
<tr><td>Notes</td><td colspan="3">[1] These are guidance only, the user should use their judgement to assess a score which reflects the standard of design and construction.

[2] Selection of score is judgment by user. Either take highest score (worst case) across both columns as giving condition (not average or minimum), or where several vulnerable features combine to give higher score. Where unsure (for example, no drawings) then do not score zero, but score most likely condition (for example, based on typical construction practice at time dam was built or upgraded).</td></tr>
</table>

Table 4.20 Supplementary guidance on current condition score for concrete gravity structure (Tier 1)

<table>
<tr><th rowspan="3">Current condition</th><th colspan="4">Extent to which feature is symptomatic of performance and thus likelihood of failure</th></tr>
<tr><th colspan="2">Seepage quantity, fines</th><th rowspan="2">Deformation</th><th>Surveillance</th></tr>
<tr><th>Dams</th><th>Service reservoirs</th><th>Monitoring</th></tr>
<tr><td>5 – Emergency drawdown</td><td>Fines being carried in seepage</td><td></td><td></td><td></td></tr>
<tr><td rowspan="3">4 – Some concern</td><td>Leaks a lot, could be carrying fines</td><td>Fails drop tests</td><td rowspan="2"></td><td rowspan="2"></td></tr>
<tr><td>Seepage increasing with time (>20% increase in a year)</td><td></td></tr>
<tr><td>Seepage > 10 times Seepage Index given in Charles et al (1986, p. 7)</td><td></td><td></td><td>Surveillance < 2 per week in dams which are vulnerable to rapid failure (Note 1)</td></tr>
<tr><td>3</td><td>Seepage quantity varies with reservoir level</td><td>No drop test results</td><td></td><td>No surveillance (dam not vulnerable to rapid failure egg backed by large embankment)</td></tr>
<tr><td rowspan="2">2</td><td rowspan="2"></td><td rowspan="2"></td><td rowspan="2"></td><td>No instruments at dam, or readings not evaluated within one week of reading</td></tr>
<tr><td>Poor ability to inspect (that is large leak would not be detected)</td></tr>
<tr><td>1 – No signs of adverse behaviour</td><td>Seepage < Index given in Charles et al. (1986, p. 7)</td><td></td><td>No differential settlement, Total settlement < Johnston et al. (1999, p. 16)</td><td></td></tr>
<tr><td>Notes</td><td colspan="4">[1] Selection of score is judgment by user. Take highest score (worst case) across all columns as giving condition (not average or minimum). Where unsure (for example, if no settlement or seepage monitoring) then do not score zero but score most likely condition.</td></tr>
</table>

5 Tier 1 – Steps 2c–f Dam breach, flood routing, consequences of failure and risk

Step 2	Risk Analysis	Step 2c	Dambreak and flood routing
		Step 2d	Consequence analysis
		Step 2e	Determine level of risk
		Step 2f	Review outputs
		Optional	Estimate range of uncertainty

5.1 Step 2c – Dam break and flood routing

The speed and depth of flooding downstream following a dam failure depends on the extent of the dam breach and the speed with which the water is released. However determining this requires detailed dam breach modelling.

A conservative assumption is to assume an instant dam failure of full dam height. Two main approaches apply for estimating the potential inundation area:

- using existing inundation maps (where available)
- undertaking a visual inspection and applying simple rules plus judgement

For a Tier 1 assessment, assume an initial (dam break) water depth at the dam of half the dam height, and follow map contours and valley slope to identify the potential inundation area. Judgement will be needed to determine how far the volume of water released from the reservoir might spread. To estimate a distance downstream, consider how the stored volume of water might disperse by looking at the downstream valley width and, say, a 0.5m depth of standing water (after the flood wave passes). A length may then be estimated that broadly matches the released volume (that is, volume = length × valley width × 0.5m depth). Limits can also be estimated by looking for locations downstream where the valley opens to a much larger river valley and hence the flood volume would rapidly disperse. A typical limit for inundation mapping would be where flood levels from the dam break fall below natural 1 in 100 year flood levels for the river valley.

See Table 19.1 for a summary of methods of breach prediction for different types of dam and Table 19.2 for a summary of methods for flood routing.

Box 5.1 Example output for Step 2c

	Comments
Inundation mapping	
	Using the Reservoir Flood maps available on the Environment Agency website
Visual inspection	
Assume an initial dam break water depth at the dam site of half the dam height and follow map contours and valley slope	19.67M
Estimate the volume of water released by the dam. Estimating the valley or floodplain surface area and potential flood water depth.	
Cascade failure	
Combine cascade volume of water that might be released by the lower dam, along with a retained water level up to 1m above the dam crest level.	n/a
Downstream limit	
How far downstream will consequences be considered?	The d/s limit has been considered to be 10km due south of the reservoir to the town of Walbourough Beck .

5.2 Step 2d – Consequences of failure

Flooding resulting from a dam break can impact on the downstream area in a variety of ways (such as injuries and fatalities, damage, disruption and loss of income) depending on the receptors located in the area of potential inundation. To account for these potential impacts the possible effects of inundation on the receptors present (identified in Step 1b) need to be evaluated.

The following method includes guidance on evaluating the effects of flooding on:

- people (including life risk)
- economic activity
- the environment
- cultural heritage

NB: If agricultural impacts have been identified as potentially significant, a Tier 2 level of assessment for this category should be considered.

5.2.1 Risk to people within the inundation area

People are potentially vulnerable to the impacts of flooding. The potential impacts on people require significant calculation of inundation and exposure. For a Tier 1 analysis the number of properties in the inundation area is used as a proxy for the designation for risk to life. (If a quantitative figure for people at risk is required then see section 9.1.1.)

Residential property within the inundation area

From the inundation map produced under Step 2c, assess the number of residential properties shown on a 1:25,000 scale map that are within the inundation area (counting individually if only a few properties but assessing broad numbers if a large number are present). Use Table 5.1 to assign a consequence magnitude (0–4) for the number of houses potentially impacted by the inundation.

It is normal to consider consequences downstream up to the point at which the dam break flood is no larger than the natural 1 in 100 chance per year fluvial flood, this either being due to attenuation of the flood wave, or reaching a point on the watercourse where other incoming tributaries increase the flood flow significantly.

This assessment may be supplemented by a site visit to verify properties exposed (normally carried out at the same time as a visit to confirm the features governing the potential extent of inundation).

Table 5.1 Assigning a qualitative value to the number of properties affected

Number of residential properties		Consequence magnitude
Level of impact for dam height <5m	**Level of impact for dam height >5m**	**Level**
None	None	0
<3	None	1
<30	<10	2
<300	<100	3
>300	>100	4

Community health/service assets

There are a variety of community service and health assets (for example, hospitals, residential care homes, fire stations, prisons and waste treatment facilities) that could be adversely affected by a flood event. Many of these are identifiable from 1:25,000 scale maps and/or from a site visit to the area. Record the consequence designation (0–4) from Table 5.2 according to the type of community health assets and services that fall within in the inundation area.

Table 5.2 Consequence designation for community health assets

Community health assets	Consequence magnitude
	Level
None	0
Any CH3 asset type	1
Residential homes Any CH1 asset type in urban area Any Water pumping & waste treatment sites Any power supply	2
Any CH1 asset type in remote rural areas	3
More than 1 asset of type CH1	4

Key:

CH1 = hospital, ambulance depot, residential home, health centre/clinic, police, fire station

CH2 = educational facility, prison, power supply (for example, transformers), water pumping and waste management sites

CH3 = pharmacies, post offices

5.1.2 Economic activity in the inundation area

Non-residential/commercial properties

From the inundation map produced under Step 2c, assess the number of commercial properties (that is, business premises/units, factories, warehouses and so on) that you can identify from a 1:25,000 scale map that are within the inundation area (counting individually if only a few properties but assessing broad numbers if a large number are present). Use Table 5.3 to assign a consequence magnitude for the number of commercial properties potentially impacted by the inundation.

Transport

Transport networks and associated assets can be damaged and/or disrupted by flooding. The impacts will depend on various factors such as the type of asset (road, railway, airport), the time of inundation, the traffic frequency/level of use, and the options for alternative routing. Consider such receptors in the inundation area that could be affected.

Record the consequence designation (0–4) from Table 5.3 according to the type of transportation assets that fall within or pass through the inundation area.

Table 5.3 Consequence designation for economic activity

Economic activity		
Non-residential/commercial properties	**Transportation assets**	**Consequence magnitude**
		Level
None	None	0
None	Any B and minor roads unless in very remote areas	1
Any retail property, factory, warehouse or office	B and minor roads in remote areas All A roads unless in remote areas	2
<10 retail properties, factories, warehouses or offices	Airports, railways, motorways A roads in remote areas	3
>10 but <20 retail properties, factories, warehouses or offices	>1 of any of airports, railways, motorways A roads in remote areas	4

5.2.3 The environment in the inundation area

Designated areas

Habitats and species can be adversely affected by flooding. Record the consequence designation (0–4) from Table 5.4 according to the type of conservation/protected area (that is, Local Nature Reserve, Site of Special Scientific Interest, Special Protection Area, Ramsar, Natura 2000 and so on) that are either wholly or partly within the inundation area.

Lists of designated areas can be sourced via the Natural England website (www.naturalengland.org.uk/ourwork/conservation/designatedareas/default.aspx).

A list of principal habitats and species of importance in England can also be found on the Natural England website (www.naturalengland.org.uk/ourwork/conservation/biodiversity/protectandmanage/habsandspeciesimportance.aspx).

The equivalent sources of information for designated areas in Scotland and Wales are the Scottish Natural Heritage website (http://gateway.snh.gov.uk/sitelink/index.jsp) and the Countryside Commission for Wales website (www.ccw.gov.uk/default.aspx?lang=en)

Table 5.4 Consequence designation for areas designated as conservation or protection areas for habitats and species

Environmental impact	Consequence magnitude
	Level
None	0
Local Nature Reserves	1
Statutory designations and designated sites/protection areas **not** containing protected habitats or species*	2
Statutory designations and designated sites/protection areas **containing** protected habitats or species*	3
Internationally designated sites (Ramsar, Natura 2000, SSSI)	4

5.2.4 Cultural heritage in the inundation area

Cultural heritage including assets such as historic buildings, parks and gardens, and ancient monuments can be damaged by floodwater. Such sites and monuments are often shown on 1:25,000 scale maps.

The National Heritage List for England can be found on the English Heritage website (http://list.english-heritage.org.uk/). The equivalent sources of information in Scotland and Wales are the Historic Scotland website (www.historic-scotland.gov.uk) and for the Historic Wales website (http://jura.rcahms.gov.uk/NMW/start.jsp).

Record the consequence designation (0–4) from Table 5.5 according to the type of cultural heritage assets located within the inundation area.

Table 5.5 Consequence designation for cultural heritage assets

Cultural heritage	Consequence magnitude level
None	0
Grade II listed buildings, registered parks and gardens	1
Grade II* listed buildings, registered parks and gardens	2
Grade I listed buildings, registered parks and gardens Scheduled ancient monuments and archaeological sites	3
UNESCO World Heritage Sites	4

5.2.5 Overall consequence of failure

Collate the consequence designations derived above into a table similar to that shown in Box 5.2.

The overall consequence class (for plotting on the matrix in Box 5.2) is obtained by summarising the impacts, and then deciding on an overall consequence class

Box 5.2 Example output for Step 2d

	Dam height	12.8m	Use Level of impact for Dam height >5mtrs
Receptor	Measure	Consequences	Comments
People	Human life (properties used as surrogate)	4	Using the inundation maps, approx 330 houses at risk
	Community health assets affected	2	Several post offices, schools, STW
Economic	Non-residential / commercial properties affected	2	Pubs, Retail outlets, Warehouses
	Transport disruption	3	A roads, motorways and a railway
The environment	Designated sites / affected areas	4	SSSI and nature reserves
Cultural heritage	Designated sites, listed buildings, scheduled monuments affected	2	Roman remains, and Egglestone Abbey
Overall consequence class used in risk matrix		4	Based on the highest consequence, human health and the environment.

(normally taken as the highest of the consequences).

5.3 Steps 2e and 2f – Determine level of risk and review

5.3.1 Step 2e – Determine level of risk

A qualitative assessment of the risk can be given by plotting the likelihood of failure (a combination of the threat – internal and external – failure mode and inundation extent) with the magnitude of potential consequences, on a simple risk matrix.

Plot the likelihood of failure (Table 4.15) and the magnitude of potential overall consequences (from Box 5.2) on a simple risk matrix. An example is presented in Box 5.3.

NB: This process can be repeated for different combinations of individual failure mode and consequence scenarios to reflect the key issues at a particular dam.

Box 5.3 Example of presenting risk scenarios for Step 2e

Likelihood of downstream flooding	Potential magnitude of consequences given downstream flooding				
	Level 0	Level 1	Level 2	Level 3	Level 4
Extreme	ALARP	ALARP	ALARP	Unacceptable	Unacceptable
Very high	Tolerable	ALARP	ALARP	ALARP	Unacceptable
High	Tolerable	Tolerable	ALARP	ALARP RISK	ALARP
Moderate	Tolerable	Tolerable	Tolerable	ALARP	ALARP
Low	Tolerable	Tolerable	Tolerable	Tolerable	ALARP
Very low	Tolerable	Tolerable	Tolerable	Tolerable	Tolerable

5.3.2 Step 2f – Review outputs

It is important to review the outputs of the consequence analysis.

- Are all important receptors accounted for in the assessment?
- Do the results look credible/realistic? (see Section 15.2.4, and additionaly for concrete dams Section 17.5.5)
- Does the analysis need revisiting or refining with better information?
- Where could it be improved?

Conduct a critical review of the outputs, considering whether it can be carried forward, or whether any of the aspects in Step 2 should be refined.

This could, for example, include the need for more accurate data, or moving to a higher tier (and hence complexity) of analysis. Key aspects to consider include:

- Are all significant consequence scenarios included?
- Are all significant dam failure scenarios considered?
- Do the risk levels look about right?
- Where are the gaps and what do I need to know more about?

5.3.3 Optional – estimate range of uncertainty

If desired you can indicate your level of confidence in the assessment of risk (the likelihood of the event and failure scenarios and the magnitude of potential consequences you undertook in Step 2e) using the categories: 'Very confident', 'Confident', or 'Not confident'.

Consider each combination of failure and consequence scenarios (that is each point you have plotted in the matrix (Box 5.3) and allocate one of the following levels of confidence that reflects your judgment.

- **Very confident:** for example, you are very confident that you have captured the likelihood of the hazard(s) and the magnitude of the consequences accurately in the risk assessment.
- **Confident:** for example, you have captured the hazards and consequences in the risk assessment but are uncertain about the likelihood of the hazards and/or the magnitude of the consequences.
- **Not confident:** for example, you are not certain that you have captured the hazards and/or the consequences sufficiently well in the risk assessment.

Record your confidence levels in a table. An example is shown in Box 5.4.

Box 5.4 Example output for Step 2f

Are all significant consequence scenarios included?	Yes		
Are all significant dam failure scenarios considered?	Yes		
Do the risk levels look about right?	Yes, although the consequences are very high and a more detailed look into this should be considered.		
What is governing total probability of failure and total consequences?	Consequences are weighted by the SSSI in the cultural heritage, which runs along the River Tee banks. The probability of failure is being dominated by the internal erosion. Detailed analysis of this threat should be considered. Report from I Carter may reduce this risk, as may a Tier 2 analysis.		
Where are the gaps and what do I need to know more about?			
Is Tier 1 appropriate	Further detailed analysis should be undertaken,		
Threat/failure mode	Confidence in hazard assessment	Confidence in consequence assessment	Comments
Internal threats			
Deterioration of dam	Confident	Very confident	
Deterioration of dam	Confident	Very confident	
Deterioration of dam	Confident	Very confident	
External threats			
Floods	Confident	Very confident	
Floods	Confident	Very confident	

6 Tier 1 – Step 3 Risk evaluation

Step 3	Risk Evaluation	Step 3a	Review tolerability of risk assessment
		Step 3b	Review options to reduce risk
		Step 3c	Proportionality
		Step 3d	Other considerations
		Step 3e	Review and make recommendations

Risk evaluation is the process of examining and judging the significance of estimated risk (that is, is the risk tolerable or not in terms of societal risk) and the consideration of what the costs and benefits are of various options to reduce the risk.

6.1 Step 3a – Review tolerability of risk

Consider the level of risk determined in Step 2e (and the level of confidence in the analysis if indicated in Table 5.3) and use Table 6.1 to determine whether risk of overall probability of failure is Tolerable, ALARP[5] or unacceptable.

Table 6.1 Presenting tolerability

Likelihood of downstream flooding	Potential magnitude of consequences given downstream flooding (ASLL)				
	Level 0	**Level 1**	**Level 2**	**Level 3**	**Level 4**
Extreme	ALARP	ALARP	ALARP	Unacceptable	Unacceptable
Very high	Tolerable	ALARP	ALARP	ALARP	Unacceptable
High	Tolerable	Tolerable	ALARP	ALARP	ALARP
Moderate	Tolerable	Tolerable	Tolerable	ALARP	ALARP
Low	Tolerable	Tolerable	Tolerable	Tolerable	ALARP
Very low	Tolerable	Tolerable	Tolerable	Tolerable	Tolerable

[5] As total safety or protection from threats such as floods cannot be guaranteed, it is common in risk management to refer to safety goals as ‘risks reduced to as low as reasonably practicable’. This is widely known as the ALARP principle. Although risks cannot be entirely eliminated, residual risk will always need to be considered, and where appropriate, mitigated consistent with the ALARP principle through the provision of other measures and instruments. For example, it may only be practicable to maintain the level of flood risk reduction to mitigate against a 1% annual probability event. If the flood risk posed by events greater than this is considered unacceptable, measures such as adaptation of the structures to accommodate overtopping, rapid drawdown, and emergency evacuation plans, may be used in order to reduce the residual risk to a tolerable level. For further information on ALARP, see Section 12.

i. If the range of the estimated total likelihood of failure in the risk matrix is above the upper line of the ALARP band then consider (a) moving on to Step 3b ('Review options to reduce risk'), or if required (b) improving the risk assessment to satisfy confidence*, or (c) conducting a Tier 2 risk analysis. (*The risk assessment may need to be improved to satisfy confidence and defensibility requirements of the owner or undertaker, and other concerned stakeholders.)

ii. If the range falls below the upper line of the ALARP band then continue to Step 3b.

Review both the build-up of likelihood of failure (Chapter 4) and build-up of consequences (Chapter 5) to understand what is governing these, and thus what is causing the intolerable risk.

HSE guidelines on tolerability of risk require consideration of both overall societal impact and individual risk. If there are population receptors close to the dam where the risk to life is significant (for example, a house where structural damage is likely due to the depth and velocity of water), then individual risk may become the determining factor in whether the risk is tolerable – even if there are more people further away at moderate or lower risk. The assessment of individual risk is inappropriate in a qualitative system. If individual risk is considered a significant issue then the user should move to a Tier 2 analysis.

Box 6.1 Example output for reviewing tolerability of risk

Threat, failure mode	Progression, failure mode	Likelihood of d/s flooding	Remarks	Potential magnitude of consequences downstream	Remarks
Deterioration of body of dam	Internal erosion of the foundation	High	Taken from Table 4.3	3	Taken from overall consequences class - the highest consequence being human health
Flood	Crest overtopping	Moderate	Taken from Table 4.4		
Flood	Instability	Moderate	Taken from Table 4.7		

Likelihood of downstream flooding	Potential magnitude of consequences given downstream flooding				
	Level 0	Level 1	Level 2	Level 3	Level 4
Extreme	Moderate	ALARP	ALARP	Unacceptable	Unacceptable
Very high	Tolerable	ALARP	ALARP	ALARP	Unacceptable
High	Tolerable	Tolerable	ALARP	IE	ALARP
Moderate	Tolerable	Tolerable	Tolerable	Flo Flo	ALARP
Low	Tolerable	Tolerable	Tolerable	Tolerable	ALARP
Very low	Tolerable	Tolerable	Tolerable	Tolerable	Tolerable

ALARP

6.2 Step 3b – Review options to reduce risk

Consider practical options exist to reduce the risk and the costs and benefits of these in terms of the reduction in risk.

i. Identify what practical options exist to reduce risk. The types of options which are normally available are summarised in Table 6.2.

ii. Estimate (qualitatively) the potential change in risk, that is, one risk class or more. (This can often mean repeating the risk assessment for a scenario where the candidate works have been completed to estimate the benefits in terms of the reduction in risk.)

iii. Estimate the associated present value total cost.

Estimates of cost, at this feasibility stage, should be increased by a factor for optimism bias, which is typically taken as 60% of project cost at feasibility stage (HM Treasury 2003, as updated in 2011). Optimism bias is the fact that experience shows that project outturn costs generally exceed the initial budget forecast by a significant amount, around 60% for feasibility estimates,. Also, present value is approximately 30 times annual cost (when using government discount rates).

Present the options and their estimated effect on risk reduction in a table. An example is shown in Box 6.2.

Table 6.2 Examples of options to reduce risk

Group of options	Examples	Comment
Reduce uncertainty in estimation of risk	Ground investigations Further studies on load, for example, flood estimation	Would allow more informed judgment in both Step 1b (failure modes identification) and estimation of probability of failure (Step 2)
Reduce the likelihood of initiation	Structural measures such as discontinuance, enlarge spillway and toe filter berm	Most reliable, but generally most costly Other examples of possible measures included in engineering guides to embankment and concrete dams and include discontinuance
Improve likelihood of detection	Enhanced surveillance/monitoring	Would reduce the probability of failure due to internal threats – see Step 2b
Reduce risk of progression	Structural to increase drawdown capacity and/or to facilitate other emergency intervention (for example, access and pump bases) Non-structural such as effective on-site plan to ensure response	Dependent on detection and human intervention to be of value
Reduce consequences	Off-site plan to ensure evacuation could be carried out quickly and effectively once warning raised	Would reduce the loss of life, where increased warning was available to those downstream. Generally not considered sufficient on their own as unlikely to be fully reliable.

Note: Some of these options affect both probability and consequences. In some cases the solution might be a combination of types/groups of options. Some options require coordination with other stakeholders.

Box 6.2 Example output of change in risk for Step 3b

		Comments	Likely reduction in risk		Change in Number Classes
			Before	After	
1	Investigations to reduce uncertainty in risk estimation	Further study on internal erosion to gain a better understanding of the likelihood of failure. Allow a more informed judgement	H	M	1
2	To reduce likelihood of initiation (structural)	-	-	-	-
3	To improve detection	Enhance surveillance and monitoring. Consider real time monitoring	VH	H	1
4	To reduce risk of progression	Effective on site plans and management of the dam	M	M	0
5	Non-structural – to reduce consequences	Off site plan to ensure evacuation could be carried out quickly and effectively once warning raised.	M	M	0

6.3 Step 3c – Proportionality

It is important to also consider (broadly) whether the costs of the measures (over a 100-year life) are proportional to the potential reduction in risk that could be achieved.

In a qualitative system it is not possible to provide a simple way of comparing costs with benefits, so where this this is an important part of the risk assessment, the user should move to a Tier 2 analysis. In simple terms, in the unacceptable zones total present value costs of the risk reduction items of millions would be proportionate if they reduced the risk by one class or more, while in the ALARP zone it is only likely to be proportionate if less than a million, and in the tolerable zone if of the order of a thousand pounds or less.

6.4 Steps 3d and 3e – Other considerations, review and recommendations

6.4.1 Step 3d – Other considerations

As well as comparing the benefits and costs of potential risk reduction measures and their proportionality, the risk assessment should consider the following questions for any practical options that can be identified to further reduce the risk.

1) **Does the risk assessment satisfy the confidence and defensibility requirements of the owner or undertaker and any other stakeholders?**

As appropriate, ensure that any societal concerns are adequately addressed. Stakeholders, including those who would be affected by dam failure or dam repairs, should be consulted and their concerns addressed. The outcomes of this evaluation can be indicated by 'Yes' or 'No', or 'A Yes' (apparent Yes) or 'A No' if the risk assessment does not satisfy the confidence and defensibility requirements of the owner or undertaker, and any other stakeholders.

2) **Have all the risk guidelines identified in the pre-assessment (see section 2.3) been adequately addressed?**

Additional risk criteria (as identified in the pre-assessment) can be listed and if appropriate the outcomes of evaluating them can be indicated using 'Yes', 'A Yes', 'A No', or 'No'.

3) **Does the dam meet published engineering standards for the UK?**

Confirm whether the dam has been assessed against published standards such as floods and seismic design, and if so did it meet the published standards, or are there outstanding deficiencies? (Refer to section 2.4.6.)

4) **Have any deficiencies identified in previous studies been addressed?**

Confirm whether the risk assessment has taken into consideration previously identified deficiencies (that are unresolved), and whether or not these have been addressed in the risk assessment.

6.4.2 Step 3e – Review and make recommendations

Where possible compare the outputs of the risk assessment with similar dams.

Bring together both the risk analysis (including the quantitative evaluation of risk reduction measures (Step 3c) and considerations of other factors (Step 3d) to make a decision recommendation with an accompanying justification (refer to the list of decision issues identified in the scoping step).

7 Tier 2 – Step 1 Risk identification

Step 1	Risk Identification	Step 1a	Failure modes identification (FMI)
		Step 1b	Identify potential consequences
		Step 1c	Review; scope risk analysis

The purpose of Steps 1a, b and c is to review and identify the potential failure modes and consequences such that the scope and focus adopted in Step 2 (risk analysis) is appropriate for the site. The stages in Step 1 should highlight key issues that will influence the risk analysis in Step 2.

7.1 Step 1a – Identification of potential failure modes

There are many ways in which dam failures can occur. Failures occur through the realisation of a combination of a threat and failure mode. Mechanisms by which failure can occur can happen rapidly or slowly overtime. The probability of failure associated with specific combinations of threat and failure mode varies; some modes of failure are more common than others.

Follow the process in Figure 7.1 to determine which failure modes should be included in the risk analysis.

- **Step i.** List all components of the reservoir system and their roles in preventing dam failure and interdependencies with other components.
- **Step ii.** Identify potential threats (initiating events) to the dam**.** List all external threats (initiating events or sources) that could initiate a failure mode. Use available knowledge of the dam, and how it and similar dams have performed in the past. Refer to the list provided in Table 7.1 but do not be limited to this list. (Further references that can assist in the identification of failure modes are shown in Table 7.2.)
- **Step iii.** Define credible failure modes. Based on the functional understanding of all components of the dam system, for each threat, consider the potential ways in which the dam could fail (that is, failure modes) that are credible (physically possible). The description of the failure mode should differentiate as a minimum:
    - threat (initiation)
    - failure mode (progression)
    - breach

Follow the process in the flow chart in Figure 7.2 to determine which failure modes are potentially credible and significant (the definitions of these terms given in the notes below the figure).

Some failure modes may involve more than one mechanism within the progression phases shown in the examples in Table 7.1.

Figure 7.1 Failure mode identification process

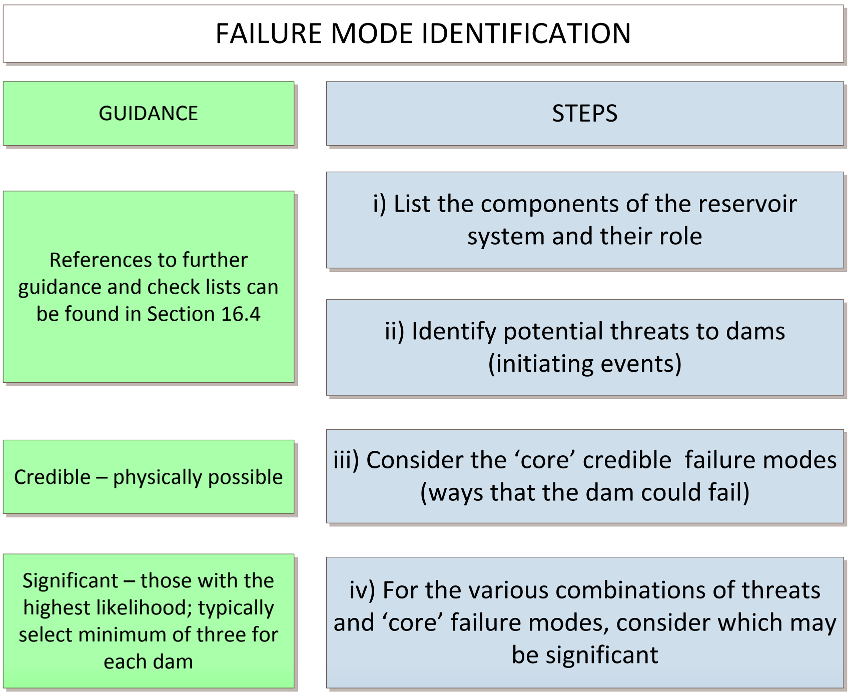


Table 7.1 Examples of mechanisms of failure mode progression

Dam type	Threat	Progression	Breach
Embankment	Flood	Overtop crest and spill down face Damage initiated to grass cover Erosion progress upstream undermines crest road which collapses	Erosion extends to waterline providing direct flow path for reservoir over erodible embankment fill
Service reservoir	Over pumping, such that Inflows> > outflow + overflow capacity	• Precast roof beams lift off at a corner such that concentrated overflow occurs onto the embankment • Perimeter embankment (which provides structural support to perimeter wall) removed by scour • Perimeter wall cracks/moves due to loss of support	Several panels in wall separate far enough to create gap of say width >50% height that is enough to release catastrophic flood wave

Further examples of possible phases within the progression phases for concrete dams and service reservoirs are shown in the event trees in Tables 8.5.9 and 8.5.10. It may be necessary to develop similar stepped descriptions to describe credible failure modes at the subject dam.

Box 7.1 Failure modes arising from lack of maintenance

In considering potential failure modes, key issues are as follows.

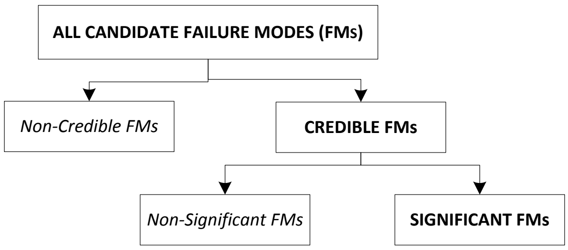
- How these relate to release of the reservoir, not damage to the dam.
- There are some types of damage, such as damage to upstream pitching, which are unlikely to lead to failure of the dam (unless the pitching were removed and the embankment eroded across the full width of the crest in a single storm), and which are likely to be noticed and remedied before damage is extensive enough to be likely to result in breach.

This guidance is based on the presumption that basic surveillance and maintenance is carried out on a regular basis, such modes of failure arising from lack of maintenance are not normally included the risk assessment. Examples of such failure modes are given below. Where lack of maintenance is an issue then the user should include these failure modes in the FMI and make their own assessment of the likelihood of failure.

Embankment dams	**Gravity dams**
• Wave erosion damage to pitching	• Blockage of foundation./ internal drains • Deterioration of reinforced concrete

- **Step iv.** Assess failure modes as significant or not significant (see Figure 7.2). List the threats considered (minimum of 10) and reasons why they are not credible/significant at a particular dam. Then select at least three of the highest probable/most significant combinations of threats and failure modes to take forward in the risk assessment.

Figure 7.2 Screening and classification of potential failure modes



1) Credible failure modes are physically possible even though they may be extremely unlikely to occur.
2) Significant failure modes: significance varies with the purpose of the risk assessment but in judging significance the cumulative effects of multiple failure modes should be considered. There should be a minimum of three, and preferably four significant failure modes for an individual dam.
3) Where uncertainty exists err on the side of safety by initially classifying a failure mode as credible and significant, and exploring the value of investigations using sensitivity analysis in the risk assessment.

NB: If insufficient evidence is available to complete this step for some potential failure modes, then either err on the side of safety by including them with the understanding that they may be excluded with further evidence and perform sensitivity studies to assess their significance and the potential value of obtaining further evidence, or obtain further evidence to the extent practicable.

Where a potential failure mode that is considered credible and significant is not listed in Table 7.2, then the user should either move to Tier 3, or develop a Tier 2 level of analysis to assess the likelihood of failure.

Further references that can assist with failure modes identification are provided in Table 16.4 in section 16.4.

Table 7.2 Matrix illustrating combinations of initiating threats and progression (failure modes)

Threat	Progression (failure mode) (code in brackets is carried forward into the Tier 2 methodology)									
	Scour	Adjacent to chute	Structural failure of		Stability failure			Liquefraction	Reactivation of fault	Internal erosion
	Downstream face		Crest wall	Chute	Body of embankment	Body of concrete dam	Dam foundation	Foundation		Four types, as ICOLD bulletin
	(Section 8.2.2) (1)	(Section 8.2.3) (2)	(Section 8.2.2) (3)	(Section 8.2.3) (4)	(Section 8.2.4) (5)	(Section 8.2.4) (6)	(Section 8.2.4) (7)	(8)	(9)	(Section 8.1.1) (10)
	External threats (guidance on quantifiying load vs. probability given in Figure 8.3									
Flood (Fl)	Overtop crest and spill over downstream face	Exceed capacity of chute		Velocity head penetrates behind stones in masonry chute	Overtop crest, fill saturates leading to slope instability	Tensile crack on lift joint	Tensile strength on foundation contact			Elevated hydraulic gradient causes hydraulic fracture
Wind (Wi)	Waves overtop crest, scour fill		Wave loading causes shear overturn		Waves overtop crest, saturate fill					
Adverse weather (Aw)	Intense rain causes local scour down mitres/other concentrated run-off				Intense rain saturates downstream slope leading to slope instability	Thick ice sheets forms on reservoir, melts fast generating large lateral load on dam causing dam instability				
Failure of upstream dam	Similar failure mode to flood (but note that failure of upstream dam should not usually be included as a threat to the downstream dam. This is because the allocated responsibility (at least the cost associated with protecting against the effects of upstream dam failure on the downstream dam) would transfer to the downstream owner. Therefore the normal convention is to attribute the consequences of downstream dam failure to the result of an upstream dam.									
Reservoir drawdown (Rds)					Slow drainage of upstream face					
Earthquake (Eq)					Seismic load causes slope instability	Seismic load causes dam instability	Seismic load causes dam instability	Seismic load	Major displacement on fault	
Uncontrolled inflow (In)	Overtop crest and spill over downstream face				Increase pore pressure in fill					
Actions of man (Ma)	Fail to open gates to pass floods				Trees removed – increase in pore pressures and loss of strength of roots. Excavation for services into toe of dam					
Deterioration	Internal threats include deterioration due to features such as stress changes due to consolidation and/or cyclic loading under seasonal changes in reservoir level. The likelihood of embankemnt dam failure due to internal threats cannot be predicted reliably by analysis, so the likelihood of this is estimated using historic failure rates – see text on interal threats.									
Body of dam (Db)					Flow through crack in core increases pwp in downstream shoulder	Degration of body (for example alkali-silica reactions) lead to loss of strength/water tightness				May be intermittent/ time related
Dam foundation (Df)							Differential settlement . stress transfer lead to sudden (brittle) failure			May be intermittent/ time related
Interface structure and fill (Di)										Commonly due to hydraulic seperation
Structural material through dam (Ds)	Pipe burst within fill supporting core/SR gravity wall erodes fill, loss of support to wall									Leak from old pipe, or through brickwork
Lining to reservoir (Li)							Leakage leads to increased pore pressure in foundation			Leakage leads to increased hydraulic gradients in foundation

CMSR = concrete/masonry dam and/or service resrvoirs
Coding = flood, leading to overtopping of crest and spill over downstream face – (Fl1)

Table 7.3 Example of a completed FMI table

Failure mode no.	Description of failure modes			Credible?	Justification	Significant?	Justification
	Initiation (threat)	Progression (failure mode)	Breach				
Internal							
FI5	High water level during flood Deterioration	Cracked core and internal erosion of embankment fill	Embankment collapse	Yes	Puddle clay core with selected fine material both sides before general fill. Chimney drain in middle of downstream shoulder of unknown grading. Unlikely to be in filter compatibility. Risk of sandstone bands in general fill	Yes	Too many unknowns. However no signs of significant settlement apart from adjacent to the spillway
FI7	High water level during flood Deterioration	Internal erosion from embankment into soil foundation	Embankment collapse	Yes	Two possible mechanisms a) clay core directly into foundation and b) downstream shoulder into foundation; sand blanket could protect but grading unknown; grading of alluvium unknown but potential for presence of sands/gravels	Yes	Too many unknowns.
FI7	High water level during flood Deterioration	Internal erosion from embankment into rock foundation	Embankment collapse	Yes	Sides of clay core at interface between general foundation stripping level and concrete cut-off is the area of risk; not certain of treatment in this area; could be a particular issue where sandstone bands intersect the core foundation	Yes	Too many unknowns.
FI1	High water level during flood Deterioration	Internal erosion at foundation surface	Embankment collapse	Yes	High permeability peat layer at interface with embankment	No	Consolidation is likely to have taken place in the 200 years since construction
FI7	High water level during flood Deterioration	Internal erosion in foundation	Embankment collapse	Yes	Concrete cut-off through foundation; as built records show extended where fault found	No	Unlikely to be a significant through 5ft thick concrete wall.
FI5	High water level during flood Deterioration	Cracking as a result of freezing of soil	Embankment collapse	No	Tarmac crest road is likely to reduce impact of frozen temperatures	No	Frozen zone less than freeboard to dam crest of 1.8m
FI5	High water level during flood Deterioration	Cross-valley differential settlement	Embankment collapse	No	Slopes are gentle and longitudinal section shows no abrupt changes in profile apart from in concrete cut-off wall		
FI5	High water level during flood Deterioration	Internal erosion of fill into culvert	Embankment collapse	Yes	Dry culvert in downstream shoulder	Yes	Culvert bulkhead is on line of the core. Upstream of core balanced water pressure. Requires defect in core to occur first in same manner as I-1. Could provide shorter path for internal erosion
FI5	High water level during flood Deterioration	Internal erosion along outside of outlet culvert	Embankment collapse	No	Concrete tunnel fully embedded in concrete cut-off; away from cut-off not clear if cast against marl or backfilled. Concrete cut-off at interface between wet and dry sections of culvert	No	Located just outside alluvium in marl; reliant on effectiveness of 5ft thick concrete cut-off

Failure mode no.	Description of failure modes			Credible?	Justification	Significant?	Justification
	Initiation (threat)	Progression (failure mode)	Breach				
Ds1	Deterioration of pipework	Pipe burst under reservoir head, escaping through joints/cracks in culvert and eroding downstream shoulder fill	Embankment collapse	Yes	Requires joints/cracks in culvert before erosion can occur	No	Culvert likely to contain flow
Ds1	High water level during flood Deterioration	Internal erosion along outside of by-wash	Collapse of spillway walls and erosion of slot through abutment	Yes	Cut-off wall extends under by-wash but upstream of road bridge; thus vulnerable area between cut-off and road bridge; base is concrete slab with open (previously bitumen filled) joints. Side walls mass concrete with rear of wall drainage; Side walls probably continuous with no joints.	Yes	However spillway is situated high up on abutment and would only lose limited depth of reservoir. Single estimate of consequences would overestimate the impact
External							
Fl1	Flood	Overtopping of crest and erosion of fill	Embankment collapse	Yes	Embankment downstream face could erode	Yes	Potential for blockage of part of bridge
Fl2	Flood	Overtopping of chute and erosion of fill	Embankment collapse	No	Chute is in cut through abutment. No access to embankment fill		
Fl5	Normal operating water levels	Slope failure and erosion either from loss of freeboard or reduction in seepage path length	Embankment collapse	Yes	Take forward for precautionary analysis	Yes	Take forward for precautionary analysis
Wi5	Wave	Saturation of downstream slope, slope failure and erosion either from loss of freeboard or reduction in seepage path length	Embankment collapse	Yes	Potential for high waves.	No	Reasonable freeboard and wave wall designed to withstand wave loading. Slope not excessively steep at 1v:2.5h
Aw5	Rainfall	Saturation of downstream slopes failure and erosion either from loss of freeboard or reduction in seepage path length	Embankment collapse	Yes	Reasonable height of slope at 12m	No	Slope not excessively steep at 1v:2.5h
Eq5	Earthquake	Settlement of crest, loss of freeboard, overtopping and erosion of fill	Embankment collapse	No	Settlement expected much less than freeboard		
Eq5	Earthquake	Failure of upstream slope, loss of freeboard, overtopping and erosion of fill	Embankment collapse	No	Slope 1v:3h with berm in higher sections. To fail water level would need to be low and will not result in freeboard being breached		
Eq5	Earthquake	Failure of downstream slope and erosion either from loss of freeboard or reduction in seepage path length	Embankment collapse	No	Factor of safety static quite high at 2.0		
	Explosion of gas main in crest	Loss of freeboard, overtopping and erosion of fill	Embankment collapse	No	Gas main surround designed to only allow blast upwards, reducing risk of breaching crest		

7.2 Step 1b – Identification of potential consequences

The area downstream of every reservoir is different. Topography, land use and occupancy all vary. Flooding of these areas will therefore result in different types and levels of impact. Level of impact will also depend upon the velocity and depth of inundation resulting from a dam failure.

The actual analysis of potential consequences of flooding is conducted in Step 2d. The purpose of looking at potential consequences at this point in the assessment is to try to gain an appreciation of the magnitude of these consequences, and also to appreciate the physical extent of potential inundation and hence data requirements for the assessment.

At Tier 2 it would be normal to consider the two failure scenarios of rainy and sunny day. Table 7.4 lists other issues that should be considered in defining the dam break and flood scenario.

Scenario	Definition of failure scenario
Sunny day	Reservoir just full (that is no flood)
Rainy day	Reservoir at level of top of crest wall (if it can withstand overtopping); or dam crest level (where there is no wall, or the wall cannot withstand overtopping) (that is a flood condition)

Table 7.4 Definition of the failure scenario for Tier 2 analysis

	Issue	Suggested normal assumption for Tier 2 analysis
1	Dam break failure scenario.	Rainy day and Sunny day
2	Exposure (distribution) of the population for example a) Night or day, working day or evening and so on b) Does hypothetical person represent whole population for example are vulnerable groups or those with short duration of exposure considered separately?	a) Time averaged over 24 hours for 365 days b) Do not differentiate vulnerable groups Typical values of exposure are given in Table 9.2, for example, 80% of people in houses
3	Allowance for warning of population at risk, and effect of shelter in reducing fatality rates	Generally no warning. Where warning is allowed, assume it only applies where there is a minimum of two hours travel time to the time that the flood wave hits the first community. Consider average fatality rate, and do not include subdivision for effect of shelter
4	Is the impact of the dam failure the total effect, or 'the incremental effect of the dam compared to no dam'?	Consider total impact only for large reservoirs For small reservoirs User decision as to whether incremental damages are considered for rainy day
5	What economic/ financial damages are to be estimated?	Property damage only
6	Other impacts	Limit to: • environmental sites with

	Issue	Suggested normal assumption for Tier 2 analysis
		international and European designations • scheduled ancient monuments • transport infrastructure • critical infrastructure, where already known to dam owner/undertaker
7	Geographical extent in which consequences are to be considered	Where peak flow in dam break flood has attenuated to be no greater than the fluvial 1 in 100 chance flood, that is it is contained within the Zone 3 flood plain

Refer to a 1:25,000 scale map of the down-valley area (and/or fluvial flood maps downloaded from the Environment Agency website – see 'What's in my backyard', reservoir flooding at www.environment-agency.gov.uk).

From the inundation map, decide the maximum extent to which consequences of dam failure are to be considered.

It is normal to consider consequences up to the point at which the dam break flood is no larger than the natural 1 in 100 chance per year fluvial flood, this either being due to attenuation of the flood wave, or reaching a point on the watercourse where other incoming tributaries increase the flood flow significantly.

It may be necessary to subdivide the map into reaches due to the following factors:

- the location of the main population potentially exposed to inundation
- any significant changes in velocity or depth (for example, if the valley widens out, or the gradient steepens)

NB: The end of a reach is normally located to include the group of properties vulnerable to inundation, such that the water depth and velocity at the end of the reach is appropriate for the property group. Lengths should be measured along the flood plain (not the watercourse).

Although a detailed calculation would define groups of people with similar exposure duration and hazard to life, for a Tier 2 analysis, where an average velocity and depth are used for each reach, then the need for a subdivision should also consider whether this is required to allow for any significant differences in depth and velocity, for example, differentiating between properties in deeper water in the centre of the valley from those in shallow water near the edge of the flood plain.

Identify the following:

- residential and commercial properties likely to be damaged/affected
- any transport infrastructure (motorways, A roads, railways) that could be affected
- any critical infrastructure (utilities assets (power, gas, water), communications assets, hospitals, police, fire and ambulance stations) that could be affected
- agricultural land that could be affected (optional where internet access is not available)

- designated environmental sites (for example, SSSIs, SACs, SPAs and LNRs) that could be affected
- cultural heritage features (for example, historic buildings, monuments and archaeological sites) that could be affected

Box 7.2 Example output of consequence identification for Tier 2, used to inform subdivision into reaches

OS Grid Ref	Km d/s of dam	Feature	Remarks
AB XXX XXX	2km	Reservoir A	Downstream reservoir
AB XXX XXX	2km	Reservoir C	Downstream reservoir
AB XXX XXX	5km	Confluence with the River Z	River confluence
AB XXX XXX	9km	Town Y, A and B roads	Town
AB XXX XXX	14km	Properties, A and B roads	Hamlet
AB XXX XXX	23km	Village Y, A and B roads	Village
AB XXX XXX	29km	Village X	Village
AB XXX XXX	34km	Suburbs, A roads and motorways	Outskirts of town
AB XXX XXX	41km	Town V, A roads	Town

7.3 Step 1c – Review of outputs and scope risk analysis

Conduct a critical review of the outputs from Steps 1a and b, considering whether they can be carried forward, or whether any aspects of the assessments should be refined. (This could, for example, include the need for more accurate evidence, or moving onto a higher tier analysis.)

For Tier 2, the output would normally comprise that shown in Table 7.5.

Table 7.5 Typical expected output from Tier 2 risk assessment

Failure mode		Consequence		Risk
Failure mode (FM)	**Partitioning of load domain**	**Scenario**	**Type**	
Credible and significant • Typically three to five FM • Plus total	Single 'dam critical' or other 'defined extreme load'	Two Rainy and sunny day (selected as appropriate for failure mode)	Numeric for a. average societal life loss (ASLL) b. individual vulnerability c. third party economic damage	Product of probability and consequences. a. See Figure 9.2 b, c – numeric value
			Single qualitative for • human health • environment • cultural heritage	A qualitative judgment, using matrix similar to the table in Box 5.2 (Tier 1)
Number of combinations provided in standard risk assessment output				
1 (combined total)	1	Worst case scenario considered with overall total probability of failure	Four measures of consequences: As 'a' to 'c' above and any 'other significant' (identified in scoping)	1 × 1 × 1 × 1 × 4 = 4 (see Table 8.15)
Notes:	[1] User can choose to display likelihood of failure of individual failure modes and/or consequences in output, where appropriate to achieving the objectives of the assessment (see section 2.2). However, the user should bear in mind the number of combinations that would be produced as the output as part of completing this scoping of risk output.			

Box 7.3 Management of uncertainty in quantitative estimates at Tier 2

Section 3.6 of Volume 1 describes the sources of uncertainty in any assessment. At Tier 2, which is a simplified quantitative methodology, estimates of the components of a risk assessment would normally be made as follows.

a) Estimates of consequences and probability of failure are single point estimates, although sensitivity analysis will often have been carried out in selecting the adopted values where it was wished to estimate upper and lower bound values of each failure mode (for example, 5% and 95% confidence limits) – this would normally mean moving to a Tier 3 analysis).

b) For external loads only, a single load is considered (for example, there are no published methods for determining upper and lower bounds on PMF estimates).

c) Evaluation of risk then considers the overall probability of failure with three measures of potential consequences of failure.

Section 21.4.4 provides commentary on the uncertainty in the analysis.

A key issue is the availability of data on the dam and their quality. The amount and nature of data required will also depend on the scenarios to be analysed. For Tier 2 analysis, at the minimum, this should include the Ordnance Survey 1:25,000 scale maps covering the inundated area (downstream valley). This should normally be supplemented by a drive down the valley at the same time as the visit to the dam.

If there are not enough data available to identify potential failure modes (Step 1a), a critical examination should be carried out of the data in relation to the failure modes for which the probability of failure is to be estimated, and a record made of what exists and its quality. (This assessment would normally be carried out by an individual, with a second carrying out quality assurance checks.)

If there is insufficient information available about the condition of the dam to inform the risk assessment then consider performing a condition assessment. See section 2.4.7 for further guidance.

8 Tier 2 – Steps 2a–b Likelihood of failure

Step 2	Risk Analysis	Step 2a	Likelihood of failure due to internal threats
		Step 2b	Likelihood of failure due to external threats

For ease of use this section presents Steps 2a and b together for embankment dams and then again for other types of dam. Embankment dams follows after the introductory section. For other dam types, go to section 8.3.

NB: The guide only provides guidance on evaluation of threats/failure modes that are commonly required at Tier 2. If additional failure modes are required reference should be made to supporting information in Part 2 and/or the methodology in Tier 3.

Threats which could lead to dam failure may be subdivided into external threats and internal threats. The latter are those where the root cause of failure is within the body of the dam, or its foundation, caused for example by deterioration or ageing. These types of failure comprise around half of the causes of failure of dams in service (Brown and Tedd 2003). This may lead directly to failure under constant load, or may weaken the dam to such an extent that it fails rapidly when subject to a change in external load. Further detail is provided in Part 2 in section 17.4.

This step (risk analysis) builds on the output from the failure modes analysis to decide which threats/failure modes are to be quantified and Step 1, which should have ensured that there are sufficient data to carry out the quantification of loads.

Box 8.1 Length effects – when should a dam be subdivided into multiple dams

The extent to which dam length needs to be considered will depend on whether the form of construction of the dam and geology vary significantly along the length of the dam. Where there is significant variability in either of these then each form of construction, or foundation geology, should be assed as a separate dam type.

Where the form of construction and geology are similar then it is a matter of judgment as to the increase in probability due to the increase length. Factors to consider are:

- the variability of the foundation (and sources of embankment fill)
- the extent to which any site specific assessment has already considered 'the worst case'
- how the long length of the dam affects the effectiveness of surveillance (and is this already factored into the probability assessment?)

8.1 Step 2a – Likelihood of failure due to internal threats for embankment dams

Internal conditions and processes within the structure of a dam can cause it to weaken or fail. The physical mechanisms controlling initiation and the rate of development of these internal threats are still not fully understood, and for internal erosion are controlled by all of the three elements of material susceptibility, stress state and hydraulic load.

Ageing of the dam and how the dam has reacted to load in the past can affect the current stress state and voids in the dam. Thus it is necessary to consider both how the dam was built (which is the 'intrinsic' condition) and its current condition. As the root cause of these effects is within the body of the dam it is difficult to measure what is happening inside the dam. Assessment therefore has to rely on external features and measurements, any monitoring of parameters within the dam, and knowledge of performance of similar dams.

8.1.1 Determining the probability of failure of embankments due to internal threats

The assessment of the probability of failure due to internal threats is still an inexact science, relying on a mixture of historical failure rates and judgement as to how these need to be adjusted for the site-specific features of an individual dam. This section therefore provides the user with a choice of methods for estimating the likelihood of failure.

It is important to note that the guide is intended for use by personnel who are familiar with assessment of the safety of dams and who therefore can apply the judgement necessary to use the methods contained here to provide reasonable estimates of annual likelihood of failure.

NB: Although historic values are provided to two significant figures, this is for consistency in scoring and does not necessarily reflect the accuracy of the values.

One of the main causes of failure of embankment dams is internal erosion, which can occur at several locations within an embankment dam or its foundation. As a minimum therefore, at least one of the internal threats in Table 8.1 should be included in any embankment dam risk assessment.

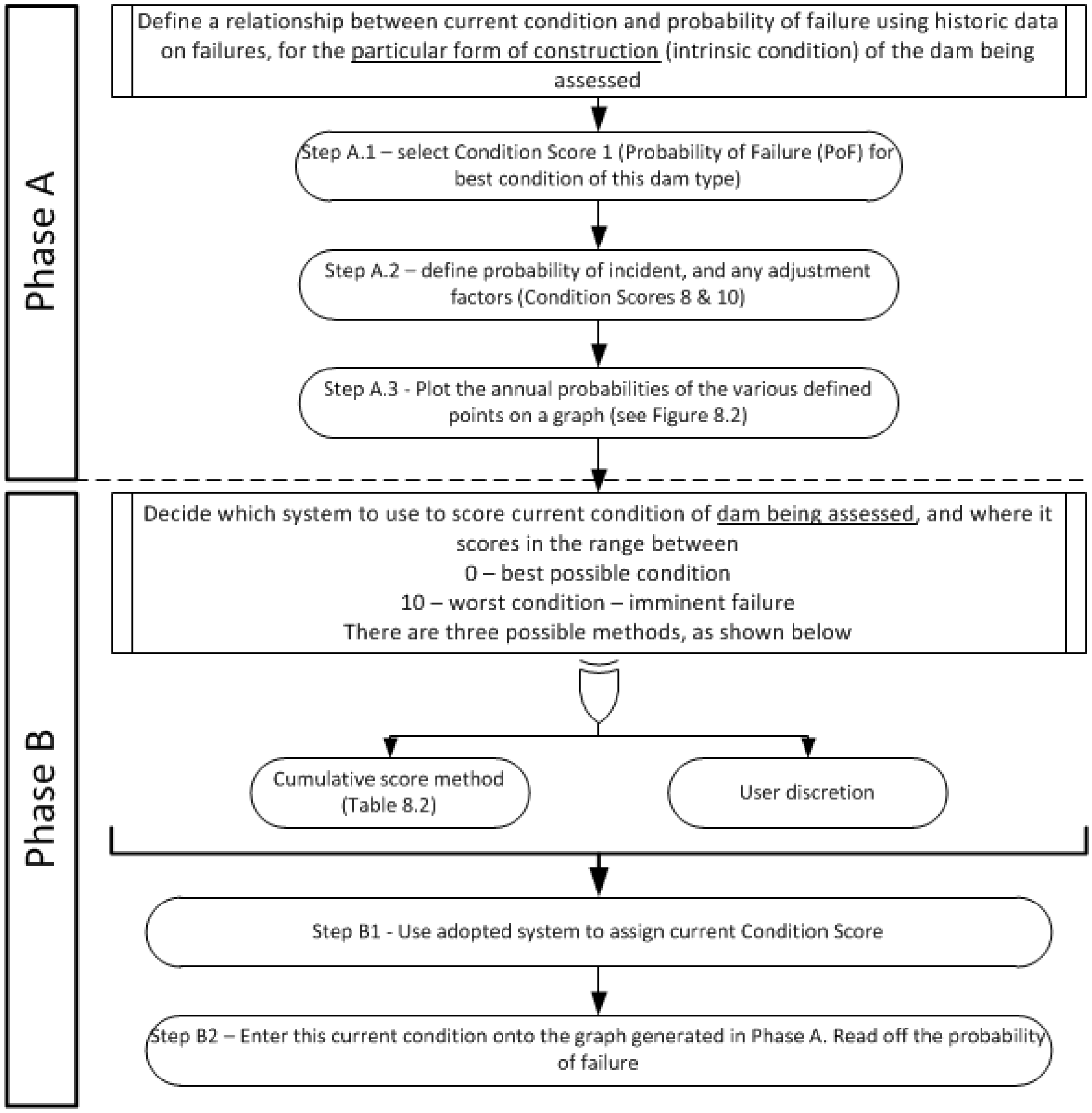
Table 8.1 Tier 2 Internal threats to embankment dams

Location of threat, as identified from failure mode identification	Method
Embankment/foundation Appurtenant works	Use data on historic frequency of incidents in UK, as given in Table 8.2, to define link of probability to Intrinsic condition, and then infer current probability of failure from current condition score
Specific location for example geological feature/ specific interface	User has three options: a. User adjustment of method above to suit individual reservoir b. Use simplified event tree described for 'dams other than embankment dams' c. Move to a Tier 3 analysis

To estimate the probability of failure due to internal threats work through the steps in Figure 8.1. The two main phases in the analysis are:

- **Phase A**: Define relationship of probability of failure to intrinsic condition, for that dam type (see guidance summarised in Table 8.2)
- **Phase B:** Assign a current condition score to the individual dam, to allow the probability of failure to be read off the output from Phase A

Figure 8.1 Flow chart of process to assess probability of failure



First, select an appropriate base probability for the three anchor points of current condition scores 1, 8 and 10, for the **type/form** of dam being assessed using the guidance in Table 8.2 and then plot the annual probability of Failure against the current condition on a graph (see Figure 8.2).

Table 8.2 Guidance on defining probability mapping for intrinsic condition for dam type/form

<table>
<tr><th colspan="2">Anchor point</th><th colspan="3">Guidance on suggested basis of values</th></tr>
<tr><th>Current condition</th><th>Description</th><th>Embankment</th><th>Buried structures</th><th>Surface structures</th></tr>
<tr><td>10</td><td>Worst condition for this form of construction; normally emergency drawdown required to avert failure</td><td colspan="3">Overall 1 in 70 based on the ratio of incidents to failures in the UK (Brown and Tedd 2003). See Table 8.16 for base probability</td></tr>
<tr><td>8</td><td>Dam in poor condition, such that works would be carried out outside a periodic safety review</td><td colspan="3">See Table 8.16 for both base probability and correction factor for dam type (no correction for appurtenant works)</td></tr>
<tr><td rowspan="3">1</td><td rowspan="3">Annual chance of failure of very good condition for this form of construction with no signs of defects or distress and a high level of both surveillance and monitoring</td><td colspan="3">See Table 8.16 for base probability. This should then be corrected for intrinsic condition of the dam as follows.

Score and then sum factors in Table 8.17 and Table 8.18 as appropriate</td></tr>
<tr><td>Cap at 15 maximum score
Divide by 1.5 to give score</td><td>Cap at 20 maximum score
Divide by 2 to give score</td><td>Cap at 5 maximum score
Score is sum of factors (capped if necessary)</td></tr>
<tr><td colspan="3">Multiply base probability by Intrinsic score x 1000/ cap defined above to give probability for anchor point 10</td></tr>
</table>

Next decide on a scoring system to obtain the current condition score for **the dam being assessed.** There are two possible methods.

- **Cumulative score method**

  a. Use the guidance in Tables 8.19 to 8.21 as appropriate to sum condition score for a variety of features. Cap score if necessary so maximum score is 10 (5 for buried structures).

  b. Add score for frequency of surveillance and speed of failure (see Box 8.2).

  c. Current condition score is then sum of the above factors.

- **User defined method.** User to define basis to obtain condition scoring. For an owner with a portfolio of dam this could be by defining some 'typical index dams' where condition and probability have been obtained from a Tier 3 analysis, with interpolation between these for the dams where detailed analysis has not yet been carried out.

Assign current condition score and use this score to read off the annual probability from the graphs generated in Phase A. An example of such a graph is shown in Figure 8.2.

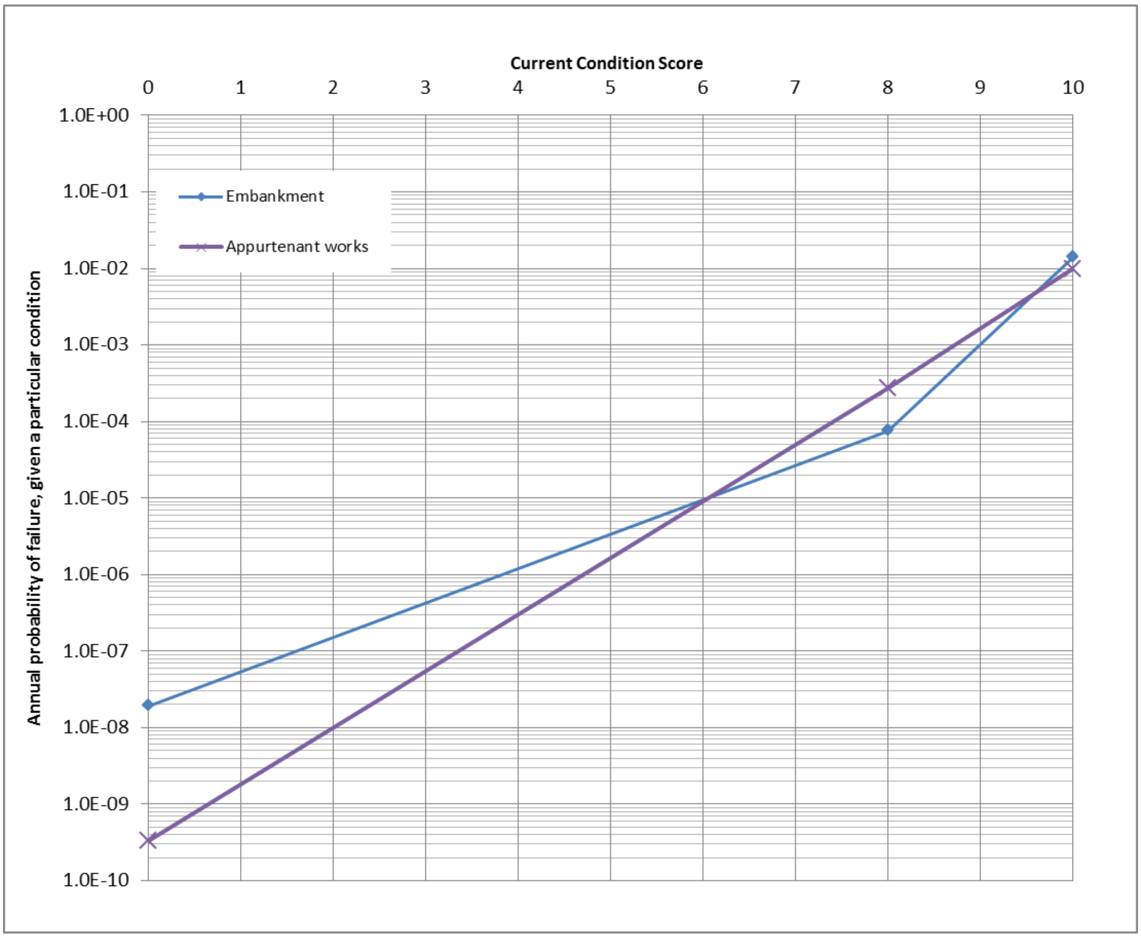
The cumulative scoring system has been devised on the basis that generally not all the indicators are adverse at the same time, so it is necessary to cap the maximum possible score. There is therefore in effect significant redundancy and where the current condition is unknown, a score of only 15% of the maximum for all the indicators would give a current condition score of 7.5. This would imply an Incident Level 3 where investigation (and if appropriate works) would normally be carried out. However, in recognition that generally some of the indicators are known, it is suggested that the proportion of marks awarded for uncertainty is as shown in Table 8.3. Application of this approach is incorporated in the guidance.

It is emphasised that the scoring is still a matter of judgement and that the initial results should be reviewed (and if appropriate adjusted) by an experienced engineer, such that the overall current condition score is a reasonable measure for the current condition of the dam, in the scale define by Table 8.2.

Table 8.3 Normal proportion of marks for varying levels of uncertainty

Degree of uncertainty	% of maximum score awarded	
	Current condition	Intrinsic condition
Unlikely (but no definitive evidence that absent)	5	25
Unknown	20	50
Likely (but not certain)	50	75

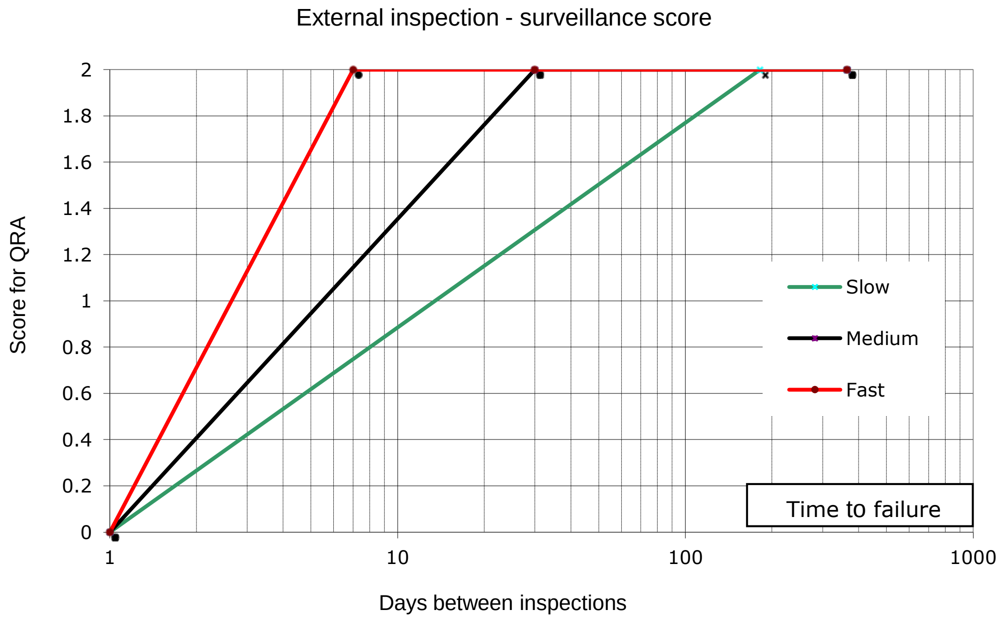
Figure 8.2 Example of a plot of annual probability of failure vs. condition score



Box 8.2 Guidance on assigning current condition given frequency of surveillance

With UK dams, surveillance has been shown to be an important feature in managing the risk of failure. Speed of failure is also an important feature and can be incorporated in the assessment. Thus the current condition score obtained from scoring system should be adjusted for frequency of surveillance and of speed of failure. (Guidance on suggested values of current condition is given in Table 8.2.)

Adjust the current condition score for frequency of surveillance using the graph below and the guidance on assessing speed of failure in Table 4.2.



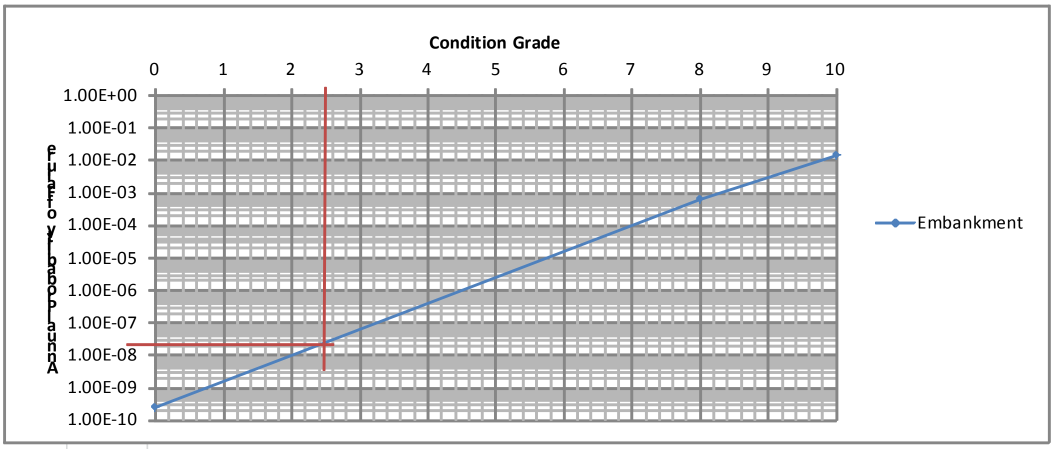
Guidance on assessing potential speed of failure of embankment dams

Feature affecting speed of failure	Fast	Medium	Slow
Embankments			
The material the watertight element is formed from	Non-cohesive soils, dispersive clays	Low plasticity clays	Highly plastic clays
Width of the watertight element	≤ H/5	Intermediate	≥H/2
Structures			
Foundation	Sandy materials	Cohesive soils	Rock
Detailing of interface of structure with embankment	Pipe in cut and cover trench, or other vulnerable interface	Intermediate	shaped to ensure positive contact stresses
Width of the watertight element	As embankment		

Note The overall assessment of likely speed of failure should be an engineering judgment, based on assessment of the following and weighted according to the individual dam.

Box 8.3 Example output for failure of embankments due to internal threats

Anchor points for Intrinsic Condition Grades - Embankment		
	Score / Value	Remarks
Condition Grade 10	1.4×10^{-2}	Taken the 1/70 for embankment dams in Table 8.2
Condition Grade 8	6.08×10^{-4}	3.8×10^{-4} in Table 8.1.2 mutiplied by the adjustment for a Puddle Clay dam, or 1.6 in Table 8.5.1B
Condition Grade 1	1.5×10^{-8}	Table 8.16, base probability 4.7×10^{-10}. Adjusted by factor up to 100 for dam features. Total score using Table 8.17 = 8 — eg....Downstream shoulder does not act as filter to core = 1, Post 1975 dam Filter in d/s shoulder = 2, Drain in shoulder but unlikely to be in filter compatibility Maximum possible score - 15 and should be divided by 1.5 to get score Adjustment = Total Score * 100 / Maximum score 8 * 100/15 = 53.33 Condition score 1 = 4.7×10^{-10} * 53.3 = 2.5051×10^{-8}
Plot the Condition Grades v Annual Probabilities		



Current Condition Grades - Embankment		
	Score / Value	Remarks
Adjustment for surveillance	1.2	Using Box 8.2 Material: Low plastic clay — Medium Width of watertight element: H / 4 — Medium Use worse case: *Medium* Frequency of surveillance: 7 days Using graph in Box 8.2. Score for adjustment = 1.2
Condition Grade	1.3	Using Table 8.19 Eg. Total sum of... ... uncontrolled seepage — 0 Seepage increasing — 0.5 Seepage carrying fines — 0.5 cont...
Condition Grade	2.5	Sum of adjustment for surveillance and condition grade
Overall probability of failure	1.8×10^{-8}	See graph above
Repeate procedure for appurtenant works where appropriate		

8.2 Step 2b – Likelihood of failure due to external threats for embankment dams

The following sections provide guidance on the likelihood of failure due to the main mechanisms driven by external threats that can lead to failure of embankment dams including:

- crest overtopping and erosion of downstream face (section 8.2.2)
- spillway chute overtopping and erosion of adjacent embankment (section 8.2.3)
- slope instability (section 8.2.4)

The likelihood of failure due to external threats to embankment dams is assessed using analytical methods. In a detailed analysis this is considered as a system response that is a range of possible responses to the possible range of load that could be applied (see Box 8.4).

However, at Tier 2 this is simplified as shown in Figure 8.3 to the likelihood of exceeding a single ("dam critical" or or other defined threshold) point load per annum and the conditional probability of failure given that load is exceeded.

Box 8.4 Selecting input parameters for risk assessment

In general a precautionary approach should be adopted by adopting conservative parameters for screening risk assessment.

Where the risk assessment shows risk is intolerable and more detailed consideration is required, then more accurate estimates, including explicit consideration of confidence limits can be made, normally using a Tier 3 analysis.

The likelihood of failure is the product of the probability of the load (threat) and the conditional probability of failure given the load.

There are a variety of ways in which the load can be selected. An example of the options for floods is shown for two cases in Figure 8.3. Case A is when the dam critical load is less than the maximum credible event and Case B is when the dam critical load is equal to or exceeds the maximum credible event (PMF). Other cases may be approarite at some dams where they may give a higher overall probability of failure, defined by the User.

Figure 8.3 Alternative methods of defining magnitude of external load

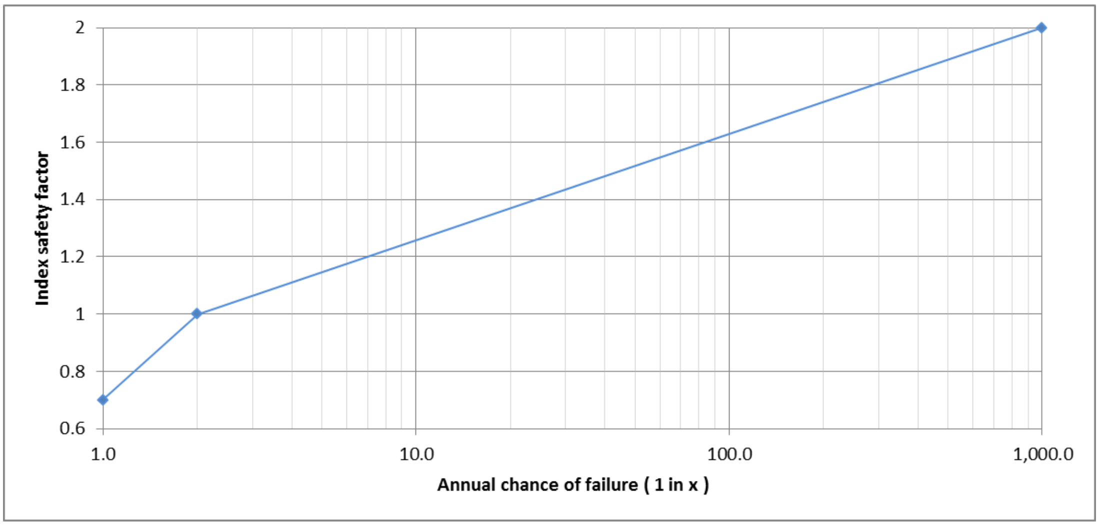
Approach to estimate the likelihood of failure due to external loads (simplified quantitative at Tier 2)

	A – Dam critical load **Dam overtops for floods < PMF**	**B – Defined extreme load for example PMF** **Dam does not overtop at PMF**
1 2	Identify the magnitude of the 'dam critical' load that would just cause failure during application of the threat for example flood inflow that would cause overtopping of crest sufficient to cause dam failure during that event.	Identify the reservoir level at PMF Estimate the safety factor on failure at the defined flood level
3	Assume conditional probability of failure = 1.0	Convert the safety factor to conditional probabilities given the load
4	Assign a probability to load (equals probability of failure since conditional probability of failure assumed = 1.0)	Estimate the AEP for the PMF (generally 2.5×10^{-6} based – see Table 8.4)
5	**The probability of failure is the product of 'probability of load' x 'conditional probability of failure given load'**	

Method 'B' in Figure 8.3 includes determination of an index safety factor using the procedure described in section 8.2.4 for embankment dams (and Section 8.3.4 for dams other than embankments) The relationship of probability of failure to factor of safety depends on the scatter in parameters used in estimating the factor of safety (due both to sampling variability and the range of safety factors that would be obtained if carrying out a Monte Carlo analysis). Further detail is provided in section 18.2.3

This has been simplified[6] to a simple single relationship as shown in Figure 8.4 based on a 50% chance of failure when the safety factor is 1.0 and a 1 in 1,000 chance of failure when the safety factor is 2.0 (noting that Figure 18.1 in section 18.2.4 suggests that a safety factor of 2 could correspond to a probability of failure between 10% (1E-2) and 1 E-5 depending on the variability of the input parameters).

Figure 8.4 Relationship of likelihood of failure to Ranking Index



8.2.1 Estimating the probability of external load

Rapid methods of load estimation can be used at Tier 2, as these are consistent with the overall accuracy of other elements of the risk assessment in this Tier. Methods for load estimation are summarised in Table 8.4. Further detail is given in Chapter 18.

[6] This calculation is simplified to a single 'dam critical load' at Tier 1 and Tier 2, but for more detailed analysis distributions of load and response can be considered. See section 18.2.3 for further information.

Table 8.4 Methods of quantification of external loads for Tier 2 risk assessment

Type of load	Method of estimation for Tier 2	Comment	Maximum credible load
Normal river flows	Use flow duration curve of daily average flow derived from that for nearest gauging station as given on internet, but adjusted pro rata on catchment area between reservoir site and gauging station	Used in estimating base flows. Cannot be used to estimate annual exceedance probabilities of peak flows.	Use floods
Floods	Generally available from previous flood studies. Where not available derive PMF using the rapid method described *Floods and Reservoir Safety* (FRS) (ICE 1996). An approximate flood frequency curve can be obtained using Table 2 in FRS and the PMF as suggested in the next column.	The annual exceedance probability (AEP) of the PMF is assigned an AEP of 1 in 400,000 (2.5×10^{-6} per year) based on plotting the FRS (1996) Table 2 relationship on lognormal probability paper and extending it to the PMF.	Flow –PMF Level – This should allow for blockage of spillway, with suggested values given in Table 8.5.
Wind generated waves	a) Follow the FRS S5 method as per the Interim Guide, OR b) Where a more detailed assessment is considered appropriate, undertake a Tier 3 analysis using the European Overtopping Manual methods, including online tools	FRS provides guidance on suggested wave freeboard for dams, based on wave height	Map of Monthly and Annual UK mean wind speeds (1971 – 2000) available at http://www.metoffice.gov.uk/
Earthquakes	The peak ground acceleration (PGA) vs. AEP relationship is taken from Table 4 of *An Engineering Guide to Seismic Risk to Dams in the United Kingdom* (Charles et al. 1991) for Zone A. The PGA is considered for a range from a threshold event of 0.05g, based on a log–log extrapolation, to exceedance of 0.375g with a characteristic magnitude M 6.5.		Maximum credible earthquake – see Charles et al. (1991)

8.2.2 Estimating the conditional probability of failure due to crest overtopping

When water levels in a reservoir are near the crest height of an embankment dam, such as following heavy rains or storms, overtopping can occur. Such overtopping can cause significant erosion and damage to the downstream face of the dam which can lead to dam failure. To prevent dams overtopping, spillway weirs are normally installed to draw off excess water from the reservoir to lower the water level. The likelihood of crest overtopping of an embankment dam under flood conditions can thus be related primarily to the capacity of the spillway relative to the flood load. The capacity of the spillway to pass the flood is also affected by its susceptibility to blockage by debris.

Undertake the following steps to calculate the magnitude and conditional probability of failure arising from flood loading leading to overweiring of the crest of the dam. An example of the calculations is provided in Box 8.6.

Establish the critical flow conditions on the downstream face that would result in failure by:

i. Calculating the duration of overtopping as a proportion of the PMF or T year storm.

ii. Use the CIRIA grass performance curves (using data from Technical Note 71; Whitehead et al. 1976) (see Figure 8.5) to extract the critical flow velocity (V_C) for grass cover failure for the given duration.

iii. Use Manning's (n value FRS chart; ICE 1996) to calculate depth on the face at V_C → flow over crest per metre.

iv. Use crest weir flow to calculate resulting reservoir levels.

v. Use reservoir levels to calculate other discharges (spillway and so on).

vi. Use FRS S6 for attenuation of reservoir inflow → inflow to reservoir to achieve dam critical conditions (for V_C m/s).

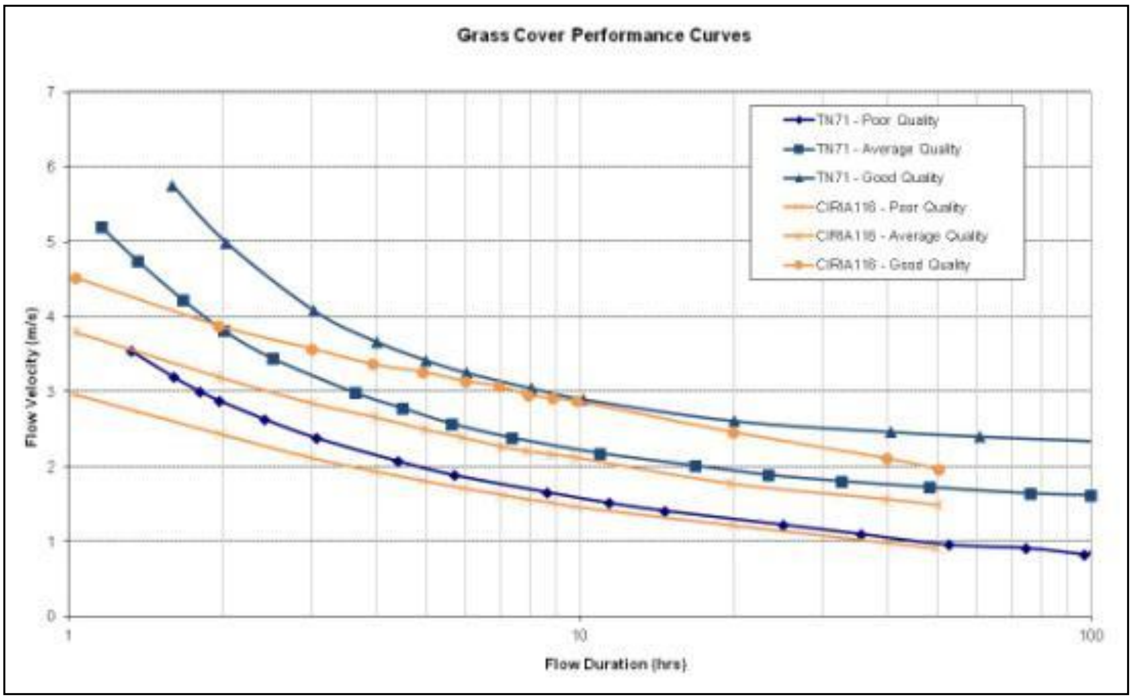
vii. Plot Y year event inflow (FRS) and identify the return period of the dam critical flood.

Box 8.5 Unit discharge due to wave overtopping

The wave run-up method recommended by ICE (1996) in *Floods and Reservoir Safety* should be used as a simple check on the adequacy of the crest level. It does not, however, give any indication as to the volume of water which will overtop the dam under wave action.

The Owen overtopping method (Owen and Steele 1988) allows the engineer to check the design crest level based on safe wave-induced overtopping passing over the embankment crest. Assessment is made depending upon the type of embankment construction and the use of the crest, for example vehicle or pedestrian access. Further guidance on wave overtopping can be found in the European overtopping manual (EurOtop 2007)

Figure 8.5 CIRIA grass performance curves



Source: Hewlett et al. (1987), Whitehead et al. (1976)

Table 8.5 gives preliminary values for blockage of spillway chutes and weirs which should be taken into consideration when using methods for the quantification of PMF loading.

Table 8.5 Preliminary values of blockage of spillways due to floating debris

Size of opening (lesser/minimum dimension)	% blockage of spillway weir/chute, where trees are present either around the reservoir or on any incoming watercourse, within 1km of the reservoir.
>10m	Nil
5–10m	10
2–5m	25
<2m	50
Notes	Where there are multiple openings (pipes or arches), adjust as appropriate (that is, either use combined width of all openings, or for larger openings assume only opening is blocked, so that percentage reduced by proportion of arches blocked to total number of arches).

Box 8.6 Example output for crest overtopping

		Score / Value	Remarks		FRS
Step i	Catchment and rainfall data	20.82 6	Area (km^2) Mainstream length, (km)		FRS Section 2.0 Catchment and rainfall data
	Slope, S_{1085} (m/km)	355 480 27.78	H_{10} (m) H_{85} (m) S_{1085} (m/km)	Identify mainstream entering reservoir (blue lin eon OS map) and measure length L (km). Estimate altitude at points 10% and 85% of length from lowest point on mainstream. Slope = H85 (m) - H10 (m) / 0.75 * Mainstream length S_{1085} = (480 - 355) / 0.75 * 6	
	Average annual rainfall, SAAR (mm)	1174	SAAR (mm)	Can be obtained from FSR Vol 5 maps	
	Average inflow q (m^3/s)	125	Assuming CWI		
	Adopt average non-separated flow, or base flow, ANSF from FSSR 16:	0.04 0.73	ANSF ($m^3/s/km^2$) q ($m^3/s/km^2$)	ANSF = [33 (CWI-125) +3.0 SAAR + 5.5] * 10^{-5} q = ANSF * Area	
	Peak of PMF inflow Q_m	221.43	Q_m (m^3/s)	Obtain Qm, in which it is assumed that the catchment soils are impermeable and that there is no urban area in the catchment $Q_m = 0.454\ \text{Area}^{0.937}\ S_{1085}^{0.328}\ \text{SAAR}^{0.319}$ $Q_m = 0.454\ (20.82)^{0.937}\ (27.78)^{0.328}\ (1174)^{0.319}$	FRS Section 3.0 Flood Peak Inflow
	PMF factor, f, *select factor from Table corresponding to appropriate return period and dam category*	1	f	Select factor from Table corresponding to appropriate return period and dam category. FRS Page 16 Eg. Cat A Reservoir, f = 1	
	Standard inflow peak, Q_i	221.43	Q_i (m^3/s)	$Q_i = Q_m * f$	
	Spillweir discharge coefficient	21.33 1.50	B (m) C		
	Initial head on spillway	0.08	m	Initial head on spillway = $(q / CB)^{2/3}$ q taken from above calculation Initial head on spillway = $(0.73 / 1.5*21)^{2/3}$	
	PMF Storm duration	16.86	PMF duration (hours)	PMF Duration = ((8 * Tp) / 60) / 60 PMF Duration = ((8 * 7585.26) / 60) / 60	
	Duration of overtopping flow-total	8.00	hours	Assumed 50% of PMF storm duration	
	Applicable to estimate critical veloc	5.33	hours	Assumed 2/3 of the duration of overtopping	
Step ii	Limiting velocity Va	3.4	m/s	Assumed good grass cover. Using Figure 8.5 CIRIA grass performance curves (Hewlett et al, 1987; Whitehead et al, 1976) to determine limiting velocity Flow duration 5.33 on graph	
Step iii	Mannings 'n'	0.02		Use manning's n (value FRS Chart page 37 Downstream slope - 2.50 Critical velocity - 3.4 m/s	
Step iv	Crest Terminal flow	0.135	m/s	Terminal flow depth on downstream face (uniform flow; wide in relation to depth, such that hydraulic radius approximately equal to depth) $nV_C / ((1 / d/s\ slope)^{0.5}))^{1.5}$ $0.023 * 3.4 / ((1 / 2.50)^{0.5})^{1.5}$	
	Unit discharge	0.459		= V_C * Crest Termainal flow =3.4* 0.135	
	Equilivant head over crest	0.306	m	Unit discharge / crest discharge coefficient Calculated crest discharge coefficient as 1.5	
	Total flow over crest	91.8	m/s	Unit discharge * crest length	
Step v	Reservoir level to give dam critical velo	58.506	mOD	Lowest point on dam crest + equilivant head over weir 58.20mOD + 0.266m	
	PMF routed outflow (Q)	144	m^3/s	Taken from the Presecribed form of record	
	PMF routed outflow at	58.057	mOD	Taken from the Presecribed form of record	
	Assumed blockage	25	%	Assumed blockage based on weir capacity Table 8.2.2	
	Spillway capacity at top of wave wall	139	m^3/s	Taken from the Presecribed form of record	
	Level of top of wave wall	59.4	mOD	Taken from the Presecribed form of record	
	Failure discharge	231	m^3/s	Spillway capacity at top of wave wall + Total flow over crest	

Continued…

….*Continued* **Box 8.6** Example output for crest overtopping

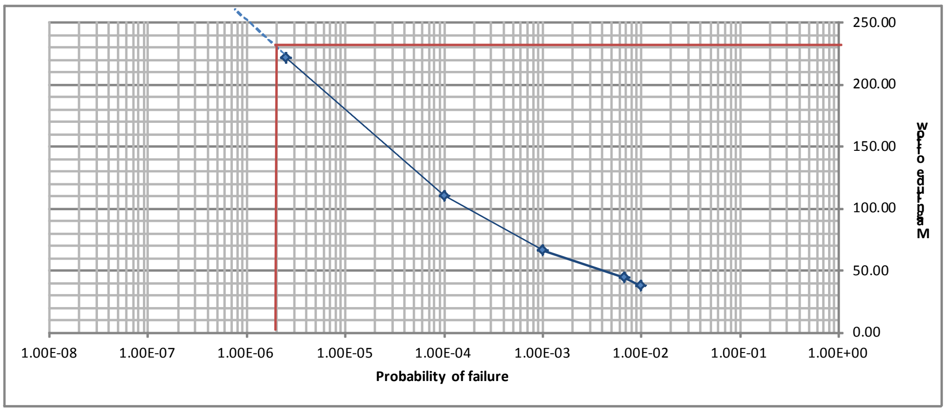
Step	Parameter	Value	Symbol	Remarks	
Step vi	Approximate mean reservoir area	1170000.00	a (m^2)	Estimate area at 0.5h above weir crest (usually probably directly from the PRF)	FRS Section 6.0 Estimated Flood Surcharge
	Head on spillway assuming no atten	3.73	H (m)	$H = Q / CB)^{2/3}$ $H = (231 / 1.5 * 21.33)^{2/3}$	
	Nature of the catchment	5300.00	K	K = 4600 Mountainous K = 5300 Hilly K = Undulatinf K = Flat	
	Time to peak of unit hydrograph (Tp)	7585.26	Tp (Seconds)	$Tp = 0.67\ KA^{0.25}$ - for PMF only $Tp = KA^{0.25}$ - for T-yeat flood or fraction of PMF	
	Storage ratio	2.59	S	$S = aH / Q_i T_p$ Where 'a' is obtained in the step above	
	Attenuation ratio	0.66	R	Obtained from Fig. 13, Page 48 in FRS with S and SAAR	
	Flood surcharge	2.5418	h	Include head due to initial flow h = RH + initial head on spillway	

Step vii

Magnitude vs. annual exceedance probability

Factor to appropriate return period	Return period (years	AEP	Equilivant fraction of PMF for rapid assessment only (f)	Q (m^3/s)	Remarks
Extrapolated from factors in FSR	PMF	2.50E-06	1	221.43	Calculate Q for each return period
	10,000 - Year	1.00E-04	0.5	110.72	Q = Routed outflow * fraction of PMF for rapid
	1,000 - Year	1.00E-03	0.3	66.43	
	150 - year	6.70E-03	0.2	44.29	Q for 10,000 year = 72 * 0.5
	100 - year	1.00E-02	0.17	37.64	Q = 72

Plot of the Magnitude vs. annual exceedance probability



AEP of failure	2×10^{-6}	Use the graph plotted above considering the failure discharge

Note:
SAAR = Standard average annual rainfall (1941-1970)
ANSF = Average non-separated flow (a FSR measure of baseflow)
CWI = Catchment wetness index

8.2.3 Estimating the conditional probability of failure due to spillway chute overtopping

Chutes channel water from the spillway weir safely to the downstream watercourse. Overflow from these structures can also erode the surface of an embankment dam.

In this guide the term critical (chute overtopping) relates to the characteristics of velocity, depth and flood magnitude that would result in overtopping of the sides of the spillway and cause sufficient erosion of the adjacent embankment for it to fail during that one flood event.

The assessment comprises a screening level assessment of the 'critical flood' by estimation of hydraulic conditions down the chute likely to correspond to the critical flood. This comprises critical flow velocity V_c on the soil slopes adjacent to the channel (the same values as for crest overweiring, provided it has the same type/quality of grass/protection to the downstream face).

The qualitative risk assessment (QRA) methodology does not consider the annual probability of failure due to structural damage to the spillway chute, for example, loss of an individual masonry block leading to a progressive and accelerating widespread loss of blocks. The vulnerability of masonry spillway to structural damage under high floods is a separate failure mode for which a defined method is not provided.

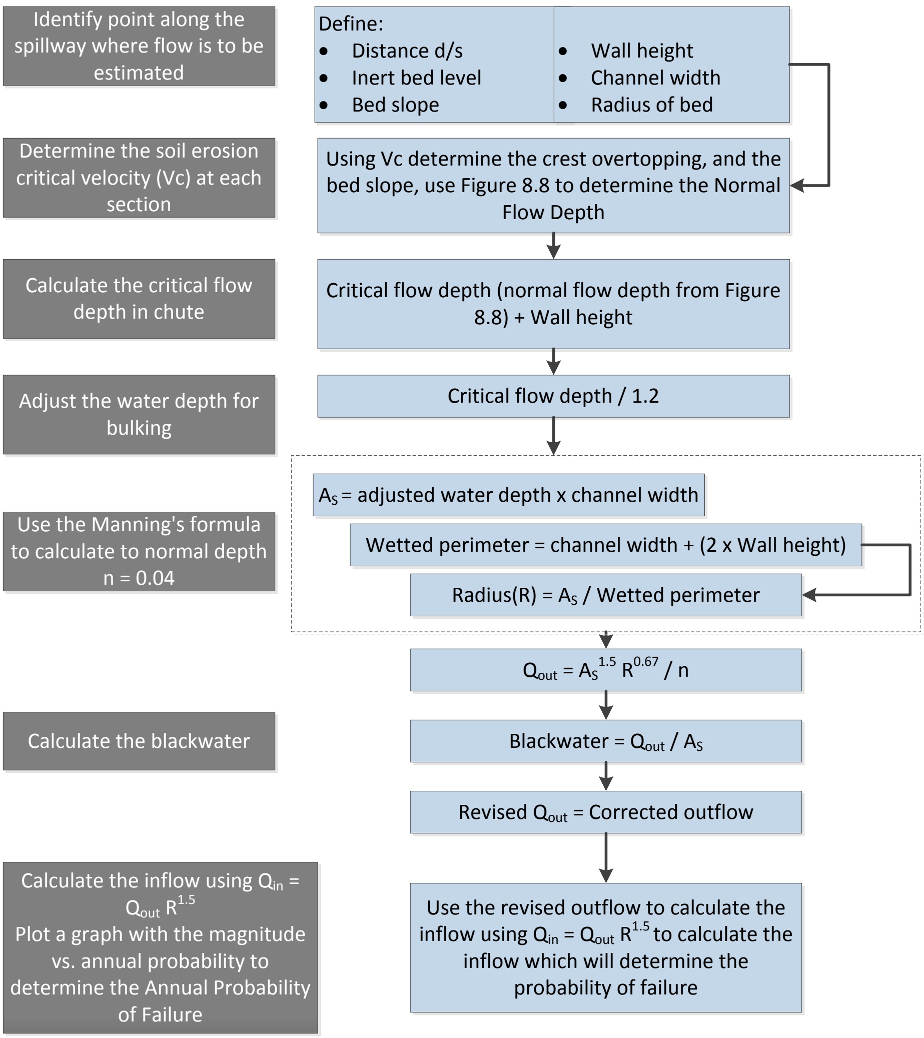
The process to assess the conditional probability of failure is set out in Figure 8.6.

The principles are as described for failure by overtopping of the dam crest. That is to assess, for a number of sections down the chute, what water level in the chute would lead to a velocity on the adjacent ground of V_c.

It does not explicitly consider the effect of a jet of water impacting directly onto the dam face, which may occur at some dams due to flow overtopping a bend (or reflected from a bend on the abutment side of the chute). The hydraulics is also simplified in calculation of average velocity in the chute by assuming glass walling of flow (that is, flow above the top of the channel is constrained to flow inside the width of the channel) to avoid the complexities of calculation of two stage flow.

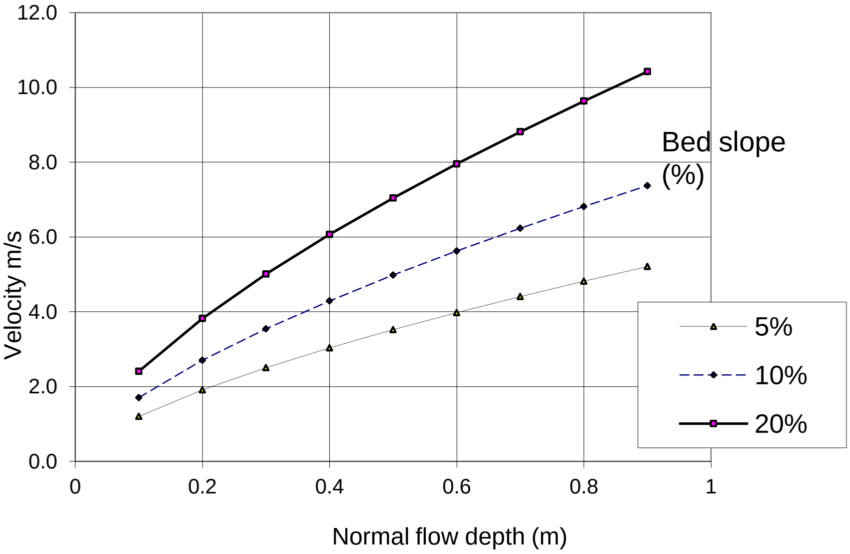
Where the spillway chute includes a bend in the channel the effect on flow and on potential overtopping can be included in the Tier 2 calculations, or transition can be made to a Tier 3 analysis. For further guidance on Tier 2 see section 18.4.2.

Figure 8.6 Steps in estimation of dam critical flood (chute overtopping)



To determine normal flow depth using *V*c from the estimation of dam critical flood, use the graph shown in Figure 8.7.

Figure 8.7 Normal flow depth for Manning's n = 0.04



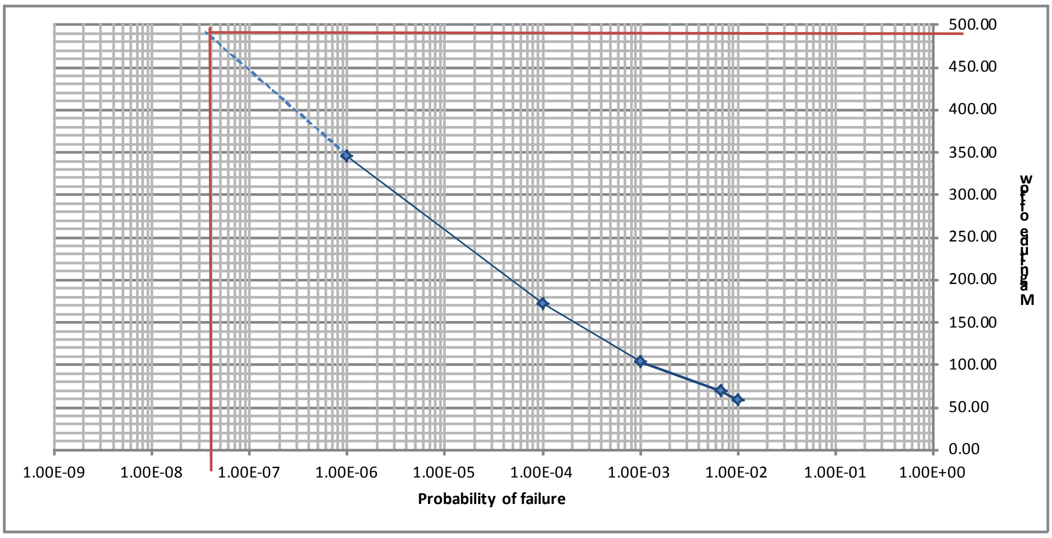
Box 8.7 Example output for spillway chute overtopping

	Location 1	Complete for each location down the spillway channel	
Inert bed level	50	mOD	Levelled on site
Bed slope	10.00	%	Measured from drawings
Wall height	1.2	mOD	On site measure
Channel width (w)	3	mOD	
Radius of Bend in horizontal alignment ®	5.4	mOD	Channel width + 2 * wall height
Dam face adjacent to top of spillway wall - critical erosion velocity V_C for each location	4	m/s	This can be calculated using the example in the Crest Overtopping
Critical depth of water above top of wall	0.35	m	Taken from Figure 8.6
Critical flow depth in chute	1.55	m	Critical depth of water above top of wall + wall height
Water depth adjusted for Bulking (air entertainment) i.e. blackwater	1.291	m	Critical flow depth in chute / 1.2
$A_S = d * w$	3.873	m	A_S = adjusted water depth * channel width
Wetted perimeter	5.4	m	Wetted perimeter = Channel width + (2 * Wall height)
Effective channel radius	0.717	m	= As / Wetted perimeter
Manning's formula, with n=0.04 $Q = A_S^{1/2}R^{2/3}/n$	39.37	m^3/s	$Q = A_S^{1/2}R^{2/3} / n$
Implied blackwater velocity	10.165	m^3/s	= Manning's formula / As
Revised Q_{out}	494.033	m^3/s	$Q_{in} = Q_{out}\ R^{1.5}$ $Q_{in} = 39.37 * 5.4^{1.5}$

Magnitude vs. annual probability

Factor to appropriate return period	Return period (years	Annual Probability	Equilivant fraction of PMF for rapid assessment only (f)	Q (m^3/s)	Remarks
Extrapolated from factors in FRS	PMF	1.00E-06	1	345.00	Calculate Q_{in} for each return period
	10,000 - Year	1.00E-04	0.5	172.50	Q_{in} = Routed inflow * fraction of PMF for rapid
	1,000 - Year	1.00E-03	0.3	103.50	
	150 - year	6.70E-03	0.2	69.00	Q_{in} for 10,000 year = 172.5 * 0.5
	100 - year	1.00E-02	0.17	58.65	Q = 72

Plot of the Magnitude vs. annual probability



Annual Probability of failure	4.0×10^{-8}	Use the graph plotted above considering the failure discharge

8.2.4 Estimating the conditional probability of failure due to slope instability

Earthquakes and ground tremors can destabilise or weaken earthen structures such as embankment dams. The threat posed by seismic action is considered to be typically small and may be ignored. However, if your dam is within a seismic risk area, then the simplified method of analysis in Appendix A to the *Application Note to for the BRE Engineering Guide to Seismic Risk of Dams in the UK* (ICE 1998) may be used.

More common in the UK is instability caused by an increase in an applied load (see Table 8.2 for examples). The probability of failure is the product of:

a) the probability of the applied load
b) the conditional probability of slope failure given that load
c) the conditional portability that the reservoir is released, given the slope instability

In principle a special case of the above is normal operation, where the probability of being full is often say 100%, such that the probability of failure is the product of 'b' and 'c'.

The key assumptions which have to be made to obtain an estimate of safety factor are summarised in Table 8.6.

Table 8.6 Key assumptions when estimating the probability of slope failure

Assumption	Typical range of values for earth dams	
Soil types	**Shear strength**	**degrees**
	Granular	30
	Clayey sand	25
	Low plasticity clay	20
	High placidity clay	15
Phreatic surface Ru = groundwater level/depth of soil	Ru of zero if slope dry Ru of 0.25 for normal operation Ru = 0.50 for full saturated slope (for example, rapid drawdown)	
External slope angle	Can be measured	
Internal zoning and foundation strata	Need to consider both zones within embankment and geology that is potential for weak layers in foundation	

Assessment of the safety factor against failure due slope instability can be carried out using one or more of the following approaches:

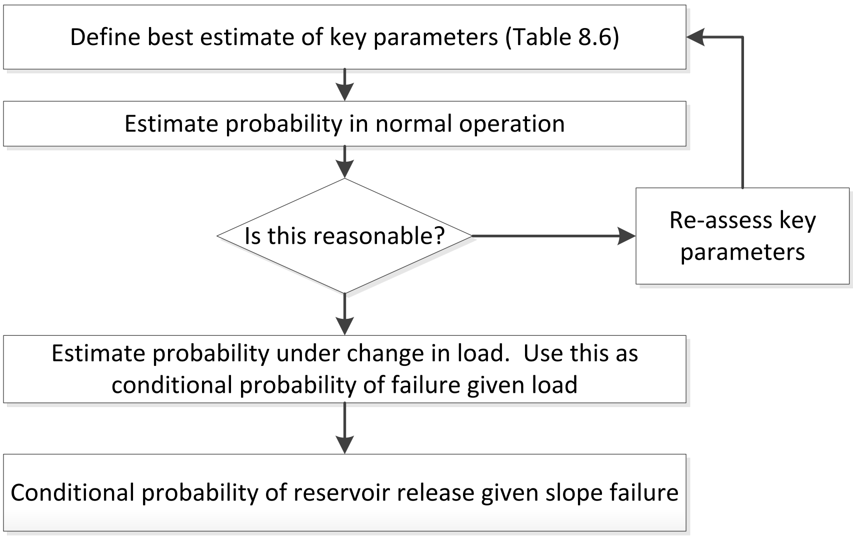
a) Compare to stability of similar slopes in the area.

b) Use simplified index charts, as order of magnitude of estimate (such charts are available as in Figure 9 of Johnston et al. (1999) and in Figures 8.9 and 8.10 below.

c) Use published stability charts such as Bishop and Morgenstern (1960) or Spencer (1967).

For all these methods one approach is to consider an estimate of the safety factor for a given set of assumptions. However, for slopes of old dams, which are often relatively steep but stable, it is often useful to back analyse the shear strength and phreatic surface required for the safety factor to be at least unity, and what reduction in safety factor would occur due to a change in loading (as Table 8.8). This process is illustrated in Figure 8.8, with stability charts in Figures 8.9 and 8.10, and guidance on the conditional probability of reservoir release given slope failure in Table 8.7.

Figure 8.8 Suggested process to assess slope instability



Box 8.8 Assessing position of phreatic surface in downstream shoulder

The only reliable way of assessing the position of the phreatic surface is to install piezometers in the downstream shoulder, with techniques described in Charles et al. (1996). Where site-specific data are not available then advice should be sought from an appropriately experienced dam engineer, with the position dependent on the type(s) of material present in the dam. Head (1982, Figure 10.14) provides a useful characterisation of permeability and drainage characteristics of the main soil types.

Where no data are available sensitivity studies using varying Ru can be used to assess the potential change in safety factor due to varying phreatic surface (which may be caused by one or more of wave overtopping/infiltration into the downstream shoulder, flow within the body of the dam over the core.

Box 8.9 What is the slope angle that should be used in the assessment?

It is important to differentiate between a slope failure of the downstream face that does not extend across a wide crest, and a slope failure which removes most of the crest and presents an imminent risk of release of the reservoir. Thus when carrying out assessment of likelihood of the dam failing and releasing the reservoir it is suggested that **the slope angle is based on the overall angle from the dam toe to the outer third of the dam crest** (for example, for 6m wide crest, to point 2m back from edge of slope). On low dams with a wide crest (for example, a public road) along the top, this will significantly reduce the slope angle considered in relation to likelihood of release of the reservoir.

Figure 8.9 Slope stability index – static

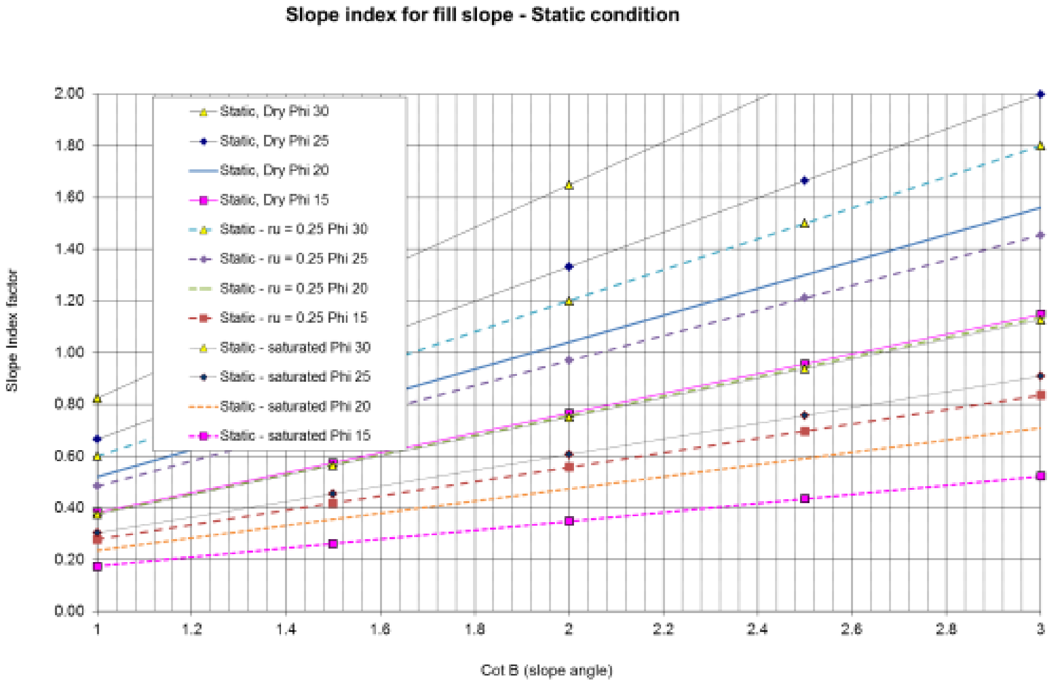


Figure 8.10 Slope stability Index – 0.28g earthquake

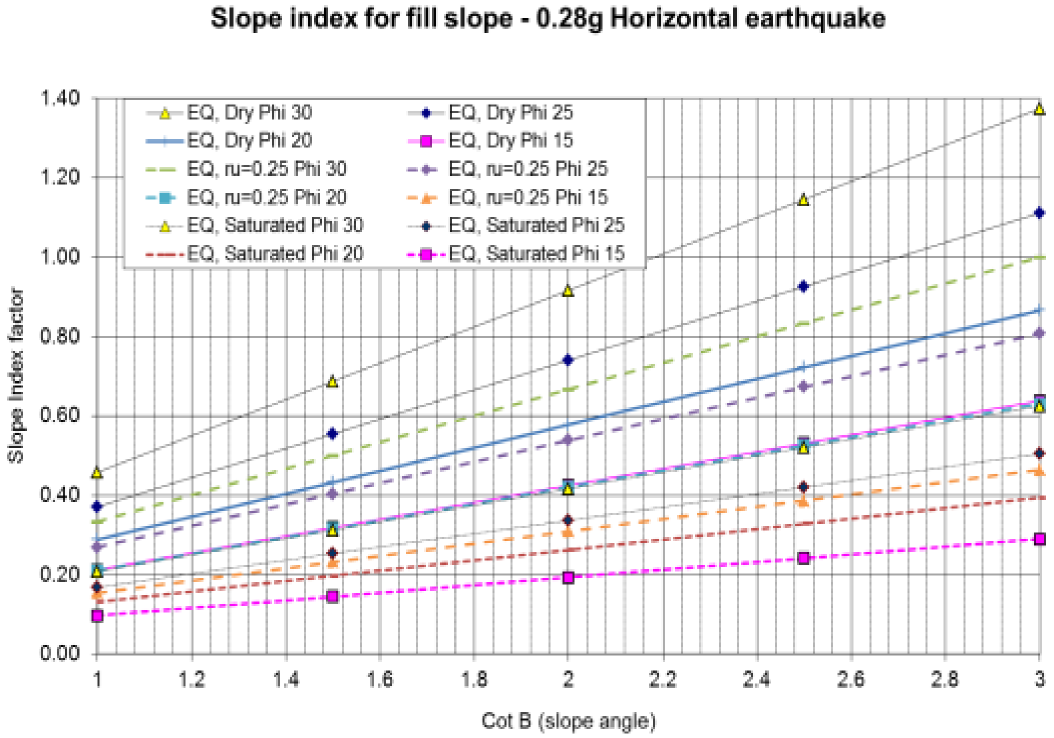


Table 8.7 Conditional probability of release of reservoir, given slope instability

<table>
<tr><td colspan="2">Base conditional probability
(correction to probability of slope failure – see flow chart in Figure 8.8)</td><td></td></tr>
<tr><td colspan="2">Median dam:
• 8m high with 1m freeboard (so water line say 3m from upstream side of crest)
• 3m wide crest, slopes as modern design and
• fill medium dense, so dilatant on shearing (not a brittle failure)
• the slip would be noticed and action initiated within three days</td><td>1 in 3,000</td></tr>
<tr><td>Factors to consider which make release of reservoir more/ less likely</td><td colspan="2">Possible impact
NB: All indicators could result in adjustments up and down from median, user to interpolate/ extrapolate from below as appropriate</td></tr>
<tr><td>Wider crest</td><td colspan="2">Less likely, say × 3 for double crest width</td></tr>
<tr><td>Reduced freeboard</td><td colspan="2">More likely, say × 0.2 freeboard <0.3m</td></tr>
<tr><td>Vertical wall along upstream edge, so water line at upstream side of crest</td><td colspan="2">Although more likely that the slip would extend into the reservoir, for release to occur the magnitude of ground movement along the slop plane must be such that the dam crest is displaced to below reservoir level. Say × 0.5</td></tr>
<tr><td>Reduced surveillance</td><td colspan="2">More likely. Say × 0.5 if likely to be one month before noticed, × 0.2 if three months before noticed</td></tr>
<tr><td>Type of fill</td><td colspan="2">Fills that are very loose, or highly plastic clays that likely to experience brittle failure are more likely to experience larger displacements and thus more likely to lead to failure of reservoir. Say maximum of × 0.1</td></tr>
</table>

Box 8.10 Example output for slope instability

	Score / Value	Remarks	
Key parameters			
Dam crest width	9	m	Taken from the PFR
Dam height	12	m	Taken from the PFR
Downstream slope 1v:	2.5h	V:H	Taken from the PFR
Downstream shoulder width	30	m	Taken from the PFR
Adding outer third of crest	33	m	Taken from the PFR
Revised downstream slope 1:v (External slope angle)	2.75h	V:H	Taken from the PFR
Soil type	25°	Phi	Clayey sand - use Table 8.6
Phreatic surface	0.25	Ru	Normal operation use Table 8.6 Ru = groundwater level / depth of soil
Slope Index Factor	1.35		Using figure 8.9
Annual Probability of failure	0.06667		1 in 15 years using Figure 8.4
Adjustment Parameters in Table 8.7		Example given in guide Base probability is 1 in 3000	
Crest width	5	m	Normal is 3m, this dam is 9m so adjusted to 5m
Freeboard	3	m	1.82 to crest, neglect wave wall, normal is 1m, adjust to 3m
Vertical wall along edge	0.7		Extends wave wall by 0.5m below crest, adjust to 0.7
Surveillance visits	0.7		Vistis every 7 days, normal is 3 days
Type of fill	1		Not expected to be loose or highly plastic take as normal
Adjustment	22050		= Base PF * Crest * Freeboard * Vertical wall * Surveillance * Fill = 3000* 5 * 3 * 0.7 * 0.7 * 1
Adjusted probability of failure	3.02×10^{-6}		Annual Probability of Failure / Adjustment

Note:
PFR = Prescribed Form of Record. A legal document that holds all key data bout the dam. Also known as the 'Blue Book'

8.2.5 Estimating the conditional probability of failure of other combinations of external threats and structural response (failure mode)

This guide has only provided detailed commentary on the most common combinations of threats and failure modes for embankment dams. References to guidance on other failure modes which may be encountered are included in Table 8.8. Other external threats would include items such as subsidence, terrorist activity and aircraft impact. These threats would not normally be considered until a Tier 3 analysis.

Table 8.8 Cross references to guidance which may be used to develop methodology for other combinations of threat and failure mode to embankment dams

Threat	Progression (failure mode)	Failure scenario (see Table 7.2)	Comments, including guidance which the user can develop into methodology to quantify annual chance of failure
Floods	High velocity flow in masonry chute displaces masonry blocks, leading to disintegration of chute	1.4	Environment Agency (2010a) Note that maximum pressure head is velocity head. McLellan (1976) provides guidance on uplift design
Floods	Elevated hydraulic gradients causes hydraulic fracture, internal erosion initiates	1.11	ICOLD Bulletin on internal erosion (ICOLD 2013)
Wind	Wave loading causes structure failure of crest wall, localised erosion of downstream face at failed section of crest wall	2.3	Methods to predict wave forces on vertical wall: • Goda (1974, 1985) - applicable to use for non-breaking waves • Takahashi et al. (1994) modification to Goda – applicable to use when a berm may cause impulsive breaking of waves • Allsop and Vicinanza (1996 –- applicable to estimate impulsive force of breaking waves • Cuomo et al. (2010b) - applicable to estimate impulsive force of breaking waves • Blackmore and Hewson (1984) – applicable to estimate force when wave action is broken before reaching the wall • Camfield (1991) – applicable to estimate force when a breaking/broken bore travels over a slope or beach
Ice	In climates and locations where ice may form, site characterization should include an assessment of the potential for ice to affect water levels due to: • build up at bridges or other structures across the spillway • build up against the embankment by wind and wave action,		Attack on revetments – see section 5.2.4.3 in CIRIA et al. (2007) USACE (2002) ICOLD (1996)

Threat	Progression (failure mode)	Failure scenario (see Table 7.2)	Comments, including guidance which the user can develop into methodology to quantify annual chance of failure
	increasing the lateral load on the embankment and freeboard required, and/or physical attack on revetments		
Upstream dam	Flood wave from failure of upstream dam leads to consequential failure of subject dam	4x	Not normally included, as failure of the upstream dam is a matter for the owner of the upstream dam, not the dam being threated. If this is to be included, then the likelihood of failure cab should be evaluated using probability of failure of the upstream dam, multiplied by the conditional probability of failure of the subject dam, given failure of upstream dam. The latter can be evaluated in the same way as for probability of failure due to floods, but with appropriate adjustment for the different incoming hydrograph.

Table 8.9 Methods of quantification of other loads (not needed in Tier 2 analysis)

Type of load	Method of estimation for Tier 2	Comment	Maximum credible load
Ice (concrete structures only)	Assume horizontal line load at water line of 100kN/m	As Hewlett et al. (2000), assuming 400mm thick ice	For screening assume that 100kN/m is max load, with annual chance of 1 in 1,000
Intense rainfall		Depending on rate of infiltration may lead to slope stability	BS EN 12056-3 for design of roof drainage shows maximum probable rainfall intensity for a two-minute duration storm on Figure NB.1 as around 0.15l/s/m2. Rainfall depth is: • 2 minute storm 1 in 10,000 chance – 12mm • 15 minute storm, 1in 10,000 chance – 16mm. Consideration should also be given to antecedent conditions.
Aircraft impact	Assume probability as function of distance from airport, as given in Thompson et al. (2001)		Fully loaded 747 freight plane, or equivalent
Mining subsidence	Should be obtained from mining	References include Highways Agency BD10/97 (design of highway structures in areas of mining	

Type of load	Method of estimation for Tier 2	Comment	Maximum credible load
	engineers overseeing the mining.	subsidence) and CIRIA SP32 (Healy 1984; being replaced by RP940)	

8.3 Step 2 - Likelihood of failure: event tree analysis for dams other than embankments

8.3.1 Introduction

Although earth embankment dams are the most numerous type of dam in the UK, there is a wide range (though a limited number) of other types of dam construction, including, concrete gravity dams (around 15% of total) and service reservoirs (around 7% of total). In some dams, there are also wide variations in the form of construction, for example, service reservoirs include brick, mass concrete, free standing reinforced concrete walls and reinforced concrete boxes where the walls are an integral part of the wall and floor. The majority rely on a gravity structure for stability, although the exact form of the structure varies and for reinforced concrete walls may include the mass of soil above the base of the wall

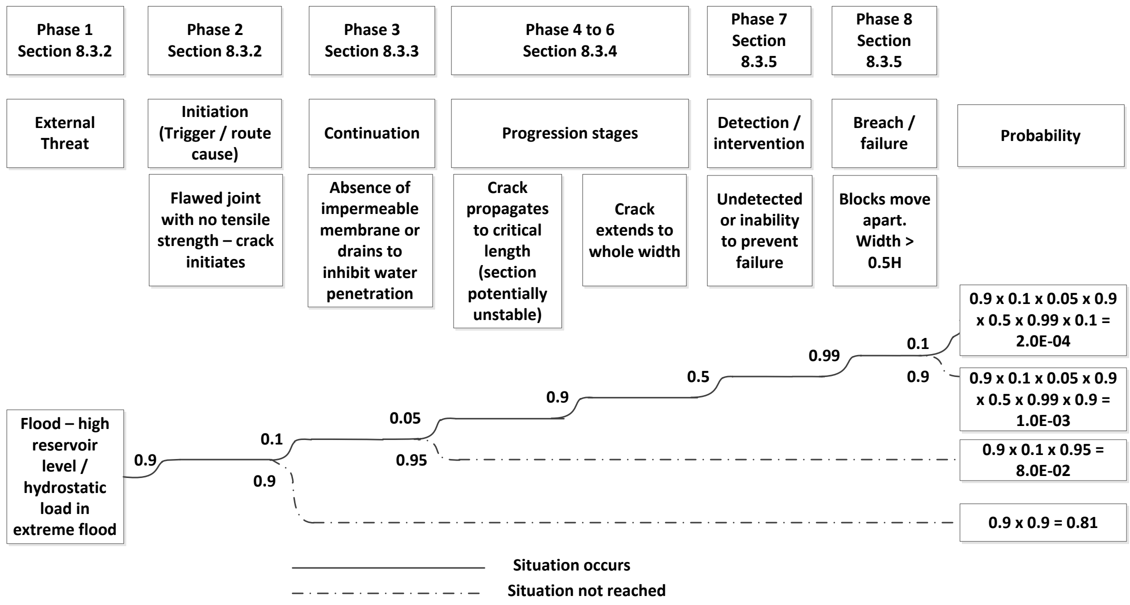
Due to the small number of each of these dam types in the UK there are insufficient data on past incidents to provide a reliable means of estimating likelihood of failure. For these gravity dams this guide provides a simplified event tree process, as shown in Figure 8.11 and as described below. Event tree templates for concrete dams and service reservoirs are provided in Tables 8.22 and 8.23 respectively. Similar trees can be constructed for other failure modes and other types of dam. For simplicity the same system is used for external threats.

Although the event tree process can be powerful in exploring and better understanding how failure progresses from initiation through to release of the reservoir, it is also more vulnerable to bias introduced by the user. This guide has therefore provided a prescribed format for event trees, derived from Chapter 24 of Reclamation's *Dam Safety Risk Analysis Best Practices Training Manual* (BPTM) (Reclamation 2010a) for internal erosion failure modes for embankment dams and application of a similar process at Loyne concrete dam (Mason 2010), as shown in Figure 8.11.

The event tree process requires a good understanding of the engineering and behaviour of dams and is should normally be carried out in a workshop process with a minimum of two, and preferably more panel engineers.

It is noted that for internal erosion failure modes for embankment dams significant work has been done based on case histories of incidents and failures and interpretation by an expert panel to support guidance for the event tree method (Fell et al. 2008a and Reclamation 2010a). However, for concrete dams similar guidance does not exist and although the event tree approach has been used with apparently reasonable results at Loyne dam and for a portfolio of 22 UK dams for one dam owner it is recommended that where the tolerability of risk is marginal and/or there is significant uncertainty then the analysis should be extended to Tier 3 methods.

Figure 8.11 Event tree process to determine probability of failure



Box 8.11 Populating event trees – description of development of phases

The description of the failure process can be broken down into a number of steps, which would vary with the failure process. However, to provide consistency with embankment dams the event trees for this screening process are broken down into eight phases from initiation, progression through to breach (see Reclamation's Best Practice Training Manual for embankment dams, Chapter 24 (Reclamation 2010a). This is the same approach as used at Loyne concrete dam (Mason et al. 2010).
It is also important to recognise that there are several alternative sequences of failure. Some common mechanisms of progression are shown in Table 8.12, with common sequences shown in the event tree templates in Tables 8.22 and 8.23.

An example is for service reservoirs where instability of the perimeter bank could trigger failure of the perimeter wall, or alternatively cracking of the perimeter wall could lead to leakage that triggers instability in the perimeter bank.

Box 8.12 Populating event trees – probability of each phase

Although guidance is provided to encourage consistency of scoring, the user should consider features at their dam which make it more or less likely for failure to progress and take this into account in scoring the probability of each step. At Tier 2, it is also important to adopt a precautionary approach where:

- if there is some uncertainty the risk analysis adopts conservative assumptions (which should be revised once more information is available)
- the scoring is based on the weakest point in the dam (where failure would initiate/occur) that is lower values in range of properties

If a failure process initiates it does not necessarily progress to complete failure. There are normally several phases the process has to go through to which probabilities of occurrence can be assigned. Figure 8.11 demonstrates these phases. Issues that should be considered when assigning likelihood of progression towards failure are explained in the following sections.

8.3.2 Reservoir level (Phase 1) and initiation (Phase 2)

For floods Phase 1 considers the annual probability of the reservoir being at or above an elevated level (underside of roof for service reservoirs) associated with the dam critical flood (or other define threshold). For loads other than floods a fraction of the time in a year that the reservoir is at or above the overflow level is typically assigned in Phase 1.

Therefore for floods, Phase 2 considers the conditional likelihood of some form of structural failure mode initiating given the reservoir level defined in Phase 1. Whereas for loads other than floods an annual probability of initiation must be assigned in Phase 2.

For floods at impounding reservoirs the Phase 1 annual probability of exceedance of peak reservoir levels can be estimated using the same process as for embankment dams as shown in Figure 8.3, and guidance on quantification of loads in Table 8.4. In addition, the stability index charts for gravity dams shown in Figures 8.12 to 8.16 can be used to identify dam critical flood levels. Estimation of the Phase 2 conditional probability of initiation is described below, while guidance on the likelihood of continuation and later phases in the event tree is given in the text on external threats in sections 8.3.3 and 8.3.4.

For loads other than floods it is suggested that the fraction of the time in a year that the reservoir is at or above the spillway crest (the level of the first high alarm for service reservoirs) should be considered, and that for this level it is very likely of the order of 99% of the time for many UK reservoirs. Where reservoir level data are available and are representative of future operation these can be used to estimate the proportion of time in a year that the reservoir is at or above the spillway crest. Table 8.10 includes guidance on estimation of the annual probability of initiation for loads other than floods. Guidance for estimating the likelihood of the water level reaching the roof in service reservoirs (just below HI alarm) can be found in Table 8.25. Service reservoirs are likely to have alarms for detection and set levels.

Table 8.10 Guidance on scoring likelihood of initiation (Phase 2) of internal threats

<table>
<tr><th rowspan="2">Threat/ FM (code in Table 7.2)</th><th colspan="2">Initiation process (likelihood of initiation in any year)</th></tr>
<tr><th>Concrete dams</th><th>Service reservoirs</th></tr>
<tr><td>Df7</td><td colspan="2">Differential settlement in dam foundation due to stress changes/ seepage leading to stability failure. (See factors in Table 8.28.)</td></tr>
<tr><td>Ds1</td><td>Pipe failure generally not significant failure mode for concrete dams</td><td>Fracture/ leak from unprotected pipe through fill, leading to erosion of fill/ stability failure
<table><tr><th>Age of pipes (years) since installation/ relining</th><th>Annual chance of initiation of defect</th></tr><tr><td><50</td><td>0.001</td></tr><tr><td>50–100</td><td>0.01</td></tr><tr><td>100–150</td><td>0.2</td></tr><tr><td>> 50</td><td>0.5</td></tr></table></td></tr>
<tr><td>Li10</td><td>Deterioration of reservoir lining generally not applicable to concrete dams</td><td>Deterioration of waterstop (or fracture due to movement) leading to significant flow through reservoir lining and initiation of internal erosion

(Judgment and BPTM pages 24-8 to 24-21)</td></tr>
</table>

Table 8.11 Guidance on scoring likelihood of initiation (Phase 2) for external threat

Threat/ FM (code as Table 7.2)	Initiation process (likelihood of initiation in any year)	
	Concrete dams	**Service reservoirs**
Fl1	Scour due to overtopping in floods generally not a significant threat to concrete dams	Roof lifts off to allow localised overtopping
Fl6, Aw6, Eq6,Db6	Joint with zero tensile strength allows crack to initiate (see Table 8.26)	
Fl7, Eq7, Df7, Li7	Crack initiates (no bond from embedment/ at rock/ concrete contact (Use index stability charts for overturning; Figures 8.14 to 8.16.)	

Box 8.13 Example output for Phase 1 - Annual Probability of Reservoir Level and 2 - Initiation

Phase 1

Root cause:	Flood
Breach failure mode description:	Instability on lift joint and in foundation

Reservoir Level

Being full at elevated flood level

High reservoir level (hydrostatic load) in extreme flood incl % blocked for SF on overturning (triangular pwp)

Calculations

Information	Units	Value/ Score	Remarks
Level of:			
Top of crest / perimeter wall	mOD	222.66	
Maximum retention level (TWL)	mOD	221.21	
Downstream bed level	mOD	204.52	
Width of:			
dam at top of wall (crest)	m	1.20	
dam at water line (A)	m	1.20	
dam at stream bed (B)	m	8.00	
Freeboard to dam crest from TWL (C)	m	1.45	= Top of dam crest - Max retention level
Freeboard to dam crest as % water depth	%	0.09	Freeboard to dam crest from TWL / water depth
Spillway:			
Crest level	mOD	221.21	
Weir width	mOD	24.00	
Weir coefficient	m	2.10	
% blockage of spillway		0.25	
Routed outflow at PMF	m^3/s	102.00	Last S10 quotes as 1.6m rise, but flow not given. Infer using Cd of 2.1

Mechanism

- Stability Index Charts
- Adjusted for foundation

Dam stability assessment using stability charts

Water level in reservoir	Units	TWL	Top of Dam	Remarks
Water depth H	m	16.69	18.14	
C/H		0.09	0	
A/H		0.07	0.07	
B/H		0.48	0.44	
Stability Index factor - lift joint				
Overturning - triangular uplift distribution (OTT)		0.95	0.8	Using Figure 8.14

Dam critical load under flood	Units	TWL	Top of Dam	Remarks
Water depth above TWL to get Overturning SF = 1.0 under traangular dist (including blockage)		-4.35		SF < 1 aat TWL is consistent with 1997 S10. Likely to have tension stress in up stream face.

Llikelihood of breach which is most likely to lead to failure (release of the	Likely
Probability using look up table 8.26	0.9

Phase 2

Root cause:	Flood
Breach failure mode description:	Instability on lift joint and in foundation

Calculations

Initiation

Some form of structural problem initiating

Likelihood of flawed joint with no tensile strength, such that crack can initiate

Mechanism - Lift joint tensile strength Crack initiates i.e. loss of bond on sides	Construction	1930 Construction, refer to Table 8.29
Llikelihood of breach which is most likely to lead to failure (release of the		Unlikely
Probability using look up Table 8.26		0.1

8.3.3 Continuation (Phase 3)

Phase 3 considers if there are any features which could prevent the defect progressing towards failure (for example, founded on non-erodible rock).

For concrete dams, the likelihood of continuation for floods leads to scour due to overtopping; this is generally not a significant threat to concrete dams.

For service reservoirs, the threat of flood can lead to erosion of supporting structural fill – depending on concentration of flow (unit discharge).

Calculate velocity and relate to allowable velocity as for crest overtopping failure of embankment dams.

For the continuation phase, stability failure in the body of the concrete dam or service reservoir can lead to water entering the lift joints. In the dam foundation, the threat to the stability of the reservoirs is unlikely unless positive features to inhibit such as:

- flexible membrane on upstream face
- upstream facing designed to be watertight for example reinforced or joints staggered from lift joints in body of dam
- internal drainage intercepts all crack flow and prevents build-up of pore water pressure
- waterbars across construction joints
- for cracks at foundation contact clay fill at heel (upstream)

Internal erosion (for example, in the foundation) is likely unless positive features such as a filter/drains allow release of water without loss of fines (but note that blocked drains may increase likelihood of failure, where they provide source of water at reservoir head to downstream part of dam).

Box 8.14 Example output for Phase 3 - Continuation

Phase 3

Root cause:	**Flood**
Breach failure mode description:	Instability on lift joint and in foundation

Calculations

Continuation Likely to continue? No feature(s) to block progression towards failure?		
No membrane or drains which inhibits water under pressure penetrating crack		
Mechanism Detailing- upstream facing, alignment of concrete facing/hearting lift joint, water stops in CJ No measures to inhibit water under pressure penetrating crack (Fine fill at heel, deep cut-off, foundation drain)	Construction	Gunite facing with A193 mesh (7mm bars at 200mm c-c, overlap 300mm min)
Llikelihood of breach which is most likely to lead to failure (release of the		Very Unlikely
Probability using look up table 8.26		0.05

8.3.4 Progression (Phases 4 to 6)

Phases 4 to 6 describe the way in which the defect can progress from 'local defect' to a widespread structural problem and the potential for a catastrophic structural collapse. There are various mechanisms with common sequences as shown in the event tree examples in Tables 8.22 and 8.23.

Table 8.12 shows some common mechanisms of progression, together with location of tools to quantify likelihood of progression (supplemented by text in this section).

As for embankment slopes, index charts are provided to enable consistent evaluation of likelihood of stability failure of gravity walls, with the charts listed in Table 8.13 and the basis of the analysis described in section 18.5.

Table 8.12 Common mechanisms of progression from initial defect to major structural problem (Phases 4 to 6)

	Mechanism of progression (subsets of predominant mode listed in Table 7.2)	**Phases which could initiate**	**Tool(s) to quantify likelihood of progression**
Perimeter earthfill bank Only an issue where the perimeter containment relies on the fill for stability			
A	Scour of supporting fill	a) Overtopping (1.1) b) Pipe burst (7.1)	**a)** Follow process for embankment dams **b)** Judgement of ratio of volumes of water released to fill eroded
B	Slope Instability of supporting fill due to: • saturation • water infilling tension cracks • earthquake (2.1) • toe excavation (3.1)	• Overtopping (1.1) • Leakage from all forms of instability/settlement/ cracks in body of perimeter wall (5.1, 8.1) • Toe excavation (3) • Aircraft breach of wall	Slope stability index (Figures 8.9 and 8.10)
Perimeter gravity wall			
C	Crack at upstream face propagates into body of dam	All forms of instability of body of perimeter wall	Concrete quality index (Table 8.5.12)
D	Failure in foundation propagates	All forms of instability of foundation of perimeter wall, for example: • crack (1.3, 2.3) • internal erosion (4.1)	Foundation quality index Table 8.28
E	Internal erosion tunnel migrates under perimeter wall	• Mechanism failure in foundation propagates • Leakage through floor (4.1)	Judgment and BPTM) (page 24-29 to 31)
F	Safety factor reduces to less than one	All forms of instability	Stability index for gravity structures (Table 8.22 and Figures 8.12 to 8.16) Basis of charts given in Table 18.8.
G	Structural movement sufficient to perforate watertight element (cracks >25mm) – see Note 1	All forms of instability	Table 8.29
Notes: [1] Further movement of the wall to an extent to allow uncontrolled release of the contents is covered by Phase 8. [2] Where a situation is not covered, or the above is considered to be inappropriate, then rely on verbal descriptors with associated likelihood (see Table 8.5.4).			

Table 8.13 Index charts for provided for Mechanism F – Instability

Figure		
Static	**0.28g EQ**	
8.6.1	8.6.2	Sliding (SLL)
8.6.3	8.6.4	Overturning, triangular distribution of pore pressures (intact lift joints) (OTT)
8.6.5	Crack does not normally remain open	Overturning, Rectangular distribution of pore pressures, cracked lift joints (and surrogate for foundation deterioration/uneven stress transfer) (OTR)
Notes:	[1] Overturning used as surrogate for development of tensile stress at upstream face Charts are for non-overflow (abutment) section – they can be adopted for use on spillway sections by setting C=O with appropriate A [3] Similarly sliding can be adjusted for use at dam/foundation interface by correcting safety factor for factor equal to assessed shear strength phi′ (c′ can be included by calculating equivalent instantaneous phi' at the appropriate normal stress). For example if an overall foundation shear strength of 40 degrees is considered representative, instead of the 45 degrees used in the charts (see Table 18.8) then the safety factor is reduced by tan 40/ tan 45 = 0.84	

Box 8.15 Example output for Phases 4 to 6 - Progression

Phases 4 to 6		Phase 4	Phase 5	Phase 6
Root cause:	**Flood**	**Calculations**		
Breach failure mode description:	Instability on lift joint and in foundation	**Progression** Describes the way in which the defect progresses from local defect to perforation of watertight element (major structural problem) with potential for catastrophic structural collapse		
		Horiz Crack propagates 50% of width	Critical crack length reached, section becomes potentially unstable in sliding or overturning	Crack extends sufficiently to extend whole width (move ds 5mm)
Mechanism		**C** *Concrete Quality Index*	**F** *Stability index (OTR)*	**G** *vertical joint details*
		1930 construction, but ongoing leakage tho dam 1941- 1978	SF< 1	No joints
Likelihood of breach which is most likely to lead to failure (release of the		Neutral	Likely	Neutral
Probability using look up table 8.26		0.5	0.9	0.5

8.3.5 Detection/intervention and breach (Phases 7 and 8)

Phase 7 considers the detection and intervention required to prevent the breach while Phase 8 considers the failure of the intervention leading to breach.

Phase 7 considers whether the defect can be detected in time for action to be taken to prevent defect progressing to collapse. Although in terms of risk, warning those downstream would reduce the consequences of failure and thus risk; this would not affect the likelihood of failure and is thus not considered further here.

Phase 8 considers the destruction level. Can the defect lead to large scale structural collapse with full uncontrolled release of reservoir? For example, with concrete dams, are the blocks interlocking, such that one cannot fail on its own, but several would have to fail together?

Guidance on the likelihood of these phases of the event tree progressing are given in Tables 8.30 and 8.31.

Box 8.16 Example output for Phases 7 and 8 – Detection/intervention and breach

Phases 7 to 8		Phase 7	Phase 8
Root cause:	**Flood**	**Calculations**	
Breach failure mode description:	Instability on lift joint and in foundation	**Unsucessful detection / intervention** likely to progress? i.e. Unsuccessful in detection/ action being taken to prevent progression?	**Breach (break through of reservoir** Defect leading to large scale structural collapse with full uncontrolled release of reservoir?
		Undetected, or even if detected unable to do anything	Movement progresses to state where blocks move apart width > 0.5H
Mechanism		leakage/ settlement + drawdown capacity	**Block width, 3D effects**
			Arch action say 90% of time would prevent fail
Llikelihood of breach which is most likely to lead to failure (release of the		Very likely	Unlikely
Probability using look up table 8.26.		0.99	0.1

8.3.6 Other failure modes

This guide only provides detailed commentary on common threats to dams other than embankments. Guidance on other failure modes which may be encountered is included in Table 8.145.

Table 8.14 Cross references to guidance for other combinations of threat and failure mode to gravity dams

Threat/ FM	Threat	Progression (Failure mode)	Comments, including guidance which User can develop into methodology to quantify annual chance of failure
Db5/6/10	Deterioration of concrete	Alkali silica reaction	ICOLD (1991)
Aw6	Ice to concrete dams	Sliding on lift joint	ICOLD (1996) USACE (2002)

8.4 Overall likelihood of failure

The overall probability of failure is useful for initial screening of dams within a portfolio, and also for screening in terms of tolerability of risk. It inevitably introduces some simplifications such as not allowing for the consequences of failure to vary with mode of failure. It also does not allow for easy identification of the measures to reduce risk, which vary with mode of failure. Where these simplifications are considered significant, either present the results by failure mode or move to a Tier 3 analysis.

Another consideration in assessing overall probability of failure is the extent to which failure modes are dependent or independent of one another. Where dependency is ignored the overall probability may be too high (although for dams the probabilities of failure are normally so small that this makes no practical difference to the outcome). At Tier 2 it is suggested that only the highest probability from each external threat is considered, but that for internal threats all failure modes are summed, as shown in Table 8.15.

In cases where several dams retain the same reservoir, but would breach into the same valley, the same approach should be taken in combining the likelihood of failure of the dams, in that only the failure mode with the highest probability for all the dams from one external threat is included, but that all internal threats are included.

Table 8.15 Example table for embankment dam of total likelihood of failure

Threat	Progression (failure mode)	Likelihood of failure for independent FM	Considered for overall probably	
			value	Comment
Floods	Crest overtopping	5E-6	5E-5	Take highest for each external threat
	Chute overtopping	5E-5		
Internal threats	Body of dam	6E-4	6e-4	Include all failure modes for internal threats
	Foundation	6e-6	6e-6	
	Interface between structure and embankment	6E-5	6e-5	
Overall likelihood of failure carried forward to Step 2e		1.8E-04	1.8e-4	

8.5 Supporting tables for Tier 2 likelihood of failure

Table 8.16 Guidance on assigning intrinsic condition scores for embankment dams

(referred to from section 8.1.1, source of values described in section 17.3.)

A. Base probabilities for dams that have been regulated under the Reservoirs Act

Condition		**Annual probability of failure due to deterioration**		**Comment**
Score	**Description**	**Embankment**	**Appurtenant work**	
10	Emergency drawdown considered necessary to avert failure	1.4×10^{-2}	1.0×10^{-2}	
8	Concern leading to works, outside periodic safety review	3.8×10^{-4}	2.7×10^{-4}	Multiply value for embankments by adjustment for dam type as table below
1	Best condition dam	4.7×10^{-10}	3.3×10^{-10}	Adjust by factor of up to 1000, for dam features which are more vulnerable to failure

B. Adjustment of annual probability for embankment failure for dam type

Dam type	**Correction to annual probability for condition score 8 (see above)**
Puddle clay	1.6
Homogenous	0.5
Rolled clay	0.2
Other	1.3

C. Base probabilities for dams that have not been regulated under the Reservoirs Act

Condition		**Annual probability of failure due to deterioration**		**Comment**
Score	**Description**	**Embankment**	**Appurtenant work**	
10	Emergency drawdown considered necessary to avert failure	3.8×10^{-2}	2.6×10^{-2}	
8	Concern leading to works, outside periodic safety review	3.3×10^{-3}	2.3×10^{-3}	Multiply by adjustment for dam type as Table B above
1	Best condition dam	Use same value as for dams regulated under the Reservoirs Act		

Table 8.17 Guidance on scoring intrinsic condition embankment dam (Tier 2)

				Fallback where no dam specific information is available: assume typical for date of construction		
Construction feature	**Max score**	**Guidance on scoring**	**Common potential failure mode(s)**	**18th century**	**19th century**	**Modern**
Buried structure						
Founded on soil	3	Score 0 if constructed as a tunnel below clay cut-off; 2 if constructed on rock with soil backfill; 3 if tunnel but passes through (spans across) clay cut-off.	Culverts surrounded by, and/or founded on soil are much more vulnerable to internal erosion along the interface	2 – pipes at base of dam	2 – generally culverts at base of valley	0 – would be tunnel in an abutment
Material through watertight element	5	Score 0 if it is concrete, or if it is surrounded in concrete, has been recently lined, is tunnel in rock 5 for wood, 3 for clay/masonry/ brickwork, 1 for metal	Some materials are more likely to crack or otherwise vulnerable to leaks	5 – wood	3 – masonry	0 – concrete
Joint type/ number	3	a) <u>General</u> – Score 0 if the joints are designed/ constructed to be watertight. Score 2 if evidence that there might be an issue with the joints. Full marks for any open joints in watertight element. 1 mark for any open joints in shoulders b) <u>Pipes</u> in fill – Score 0 if the pipe has been lined with a plastic liner, for example. Score 1.5 if the construction methods of the time suggest that the joints could have opened (for example spigot and socket).	Some dams have joints or openings within the culvert which could allow ingress of fines	3- open jointed pipes	1.5 – pipe joints sealed but vulnerable to opening	0 – no open joints
Pipe under reservoir pressure in direct contact with fill	10	Full marks for in 'contact with downstream shoulder materials'. Half marks for in 'contact with watertight element'. 50% of above where normally not under reservoir pressure	Pipes containing water at full reservoir head are vulnerable to leakage into the fill, the significance depending on the location of the potential leaks	Not applicable as can be seen/ established from visual inspection		
No downstream filter or filter zone round conduit	3	This is a specific filter around the culvert, usually only seen in modern dam construction. Score 0 if the outlet structure is in a tunnel	A filter around the culvert would reduce vulnerability to internal erosion along the outside of the culvert	3	3	0 – sand filter around outside of culvert

				Fallback where no dam specific information is available: assume typical for date of construction		
Construction feature	**Max score**	**Guidance on scoring**	**Common potential failure mode(s)**	**18th century**	**19th century**	**Modern**
Outlet conduit in deep and narrow trench (depth > width); or at toe of steep abutment	2	Only score 0 where no risk of local differential settlement over outlet/ no local reduction in vertical stress in vicinity of outlet.	Steep sided valleys or excavations are more likely to have stress transfer across the edge, leading to localised low vertical stress and thus increased vulnerability to hydraulic fracture.	2 – no attempt to reduce stress concentrations over outlet	2 – assume no attempt to reduce stress concentrations over outlet	0 – Tunnel
Poor conduit geometry that is features that make compaction around the conduit difficult	2	If there is evidence of concrete hunching score 0. If there is evidence of construction in a rock trench and the backfilling with mortar, score 0. Score 1 when vertical sided that is no overhang.	Poor compaction would lead to increased vulnerability to internal erosion along the interface.	2 – pipes with loose fill under haunches	2 – masonry culvert with vertical side walls	0 – Tunnel
Location of control (of reservoir head)	10	10 for downstream, 5 for upstream, 0 for at watertight element. For tunnels in rock score as nil. For pipes within culverts score half values shown in guidance.	The upstream control will mean there is a pressure difference between the saturated upstream shoulder and the empty outlet. This could result in leakage into the outlet culvert. The converse is true for downstream control.	Not applicable as can be seen/ established from visual inspection		
Number of means of control on each draw off	2	Where several inlets score on worst case inlet. 2 for single control; 0 for 2 valves	Two valves at the inlet provide redundancy in case of problems.	Not applicable as can be seen/ established from visual inspection		
Surface structure (for example, spillway)						
Founded on soil	3	If founded wholly on rock then score 0; if on weathered rock on abutment score 1	If founded on soil more vulnerable to concentrated leakage under foundation.	Not applicable as can be seen/ established from visual inspection		
Located on embankment	3	If on the abutment score 0. If any part of the structure is on the embankment score full marks	Any leakage into/along/out of structure is more likely to lead to a failure of the embankment.	Not applicable as can be seen/ established from visual inspection		
Detailing of interface with embankment/ abutment poor (likely to lead to seepage/ low contact stresses at junction with watertight element)	2	From discussions with the Supervising Engineer. Consider the extent of the clay core and whether there is a cut off extended under the spillway channel too.	High vertical walls and/or separate backfill to excavation for the structure are weak points where concentrated leakage could occur, leading to internal erosion and failure.	2	2	0 – back of walls battered to reduce risk of low contact stresses

				Fallback where no dam specific information is available: assume typical for date of construction		
Construction feature	**Max score**	**Guidance on scoring**	**Common potential failure mode(s)**	**18th century**	**19th century**	**Modern**
No downstream filter or filter zone at interface with embankment	1	Modern dams would have a filter around the structure, rear wall drainage along the back of the spillway walls, and so on These designs would score 0.	Particle migration associated with concentrated leakage will not be stopped, allowing on-going erosion leading ultimately to failure.	1	1	0 – filter behind clay core extended to act as filter to any leakage along the contact between the structure and adjacent fill
Inadequate tail water	2	Design for < 25% of FRS Design Flood. Consider the size of the tail water structure. If the outfall is a long way from the embankment toe, then score 0 here	Where the tail water is inadequate scour will occur, leading to undermining and ultimately collapse of the structure.	2	2	0

Table 8.18 Intrinsic condition appurtenant works

Construction feature	Max score	Guidance on scoring	Common potential failure mode(s)	Fallback where no dam specific information is available: assume typical for date of construction: 18th century	19th century	Modern
Buried structure						
Founded on soil	3	Score 0 if constructed as a tunnel below clay cut-off; 2 if constructed on rock with soil backfill; 3 if tunnel but passes through (spans across) clay cut-off.	Culverts surrounded by, and/or founded on soil are much more vulnerable to internal erosion along the interface.	2 – pipes at base of dam	2 – generally culverts at base of valley	0 – would be tunnel in an abutment
Material through watertight element	5	Score 0 if it is concrete, or if it is surrounded in concrete, has been recently lined, is tunnel in rock 5 for wood, 3 for clay/masonry/ brickwork, 1 for metal	Some materials are more likely to crack or otherwise vulnerable to leaks.	5 – wood	3 – masonry	0 – concrete
Joint type/ number	3	a) General – Score 0 if the joints are designed/ constructed to be watertight. Score 2 if evidence that there might be an issue with the joints. Full marks for any open joints in watertight element. 1 mark for any open joints in shoulders b) Pipes in fill – Score 0 if the pipe has been lined with a plastic liner, for example. Score 1.5 if the construction methods of the time suggest that the joints could have opened (for example spigot and socket).	Some dams have joints or openings within the culvert which could allow ingress of fines.	3 – open jointed pipes	1.5 – pipe joints sealed but vulnerable to opening	0 – no open joints
Pipe under reservoir pressure in direct contact with fill	10	Full marks for in 'contact with downstream shoulder materials'. Half marks for in 'contact with watertight element'. 50% of above where normally not under reservoir pressure.	Pipes containing water at full reservoir head are vulnerable to leakage into the fill, the significance depending on the location of the potential leaks.	Not applicable as can be seen/ established from visual inspection		
No downstream filter or filter zone round conduit	3	This is a specific filter around the culvert, usually only seen in modern dam construction. Score 0 if the outlet structure is in a tunnel.	A filter around the culvert would reduce vulnerability to internal erosion along the outside of the culvert.	3	3	0 – sand filter around outside of culvert

				Fallback where no dam specific information is available: assume typical for date of construction		
Construction feature	**Max score**	**Guidance on scoring**	**Common potential failure mode(s)**	**18th century**	**19th century**	**Modern**
Outlet conduit in deep and narrow trench (depth > width); or at toe of steep abutment	2	Only score 0 where no risk of local differential settlement over outlet/ no local reduction in vertical stress in vicinity of outlet.	Steep sided valley or excavation are more likely to have stress transfer across the edge, leading to localised low vertical stress and thus increased vulnerability to hydraulic fracture.	2 – no attempt to reduce stress concentrations over outlet	2 – assume no attempt to reduce stress concentrations over outlet	0 – tunnel
Poor conduit geometry that is features that make compaction around the conduit difficult	2	If there is evidence of concrete hunching score 0. If there is evidence of construction in a rock trench and the backfilling with mortar, score 0. Score 1 when vertical sided that is no overhang.	Poor compaction would lead to increased vulnerability to internal erosion along the interface.	2 – pipes with loose fill under haunches	2 – masonry culvert with vertical side walls	0 – tunnel
Location of control (of reservoir head)	10	10 for downstream, 5 for upstream, 0 for at watertight element. For tunnels in rock score as nil. For pipes within culverts score half values shown in guidance.	The upstream control will mean there is a pressure difference between the saturated upstream shoulder and the empty outlet. This could result in leakage into the outlet culvert. The converse is true for downstream control.	Not applicable as can be seen/ established from visual inspection		
Number of means of control on each draw off	2	Where several inlets score on worst case inlet. 2 for single control; 0 for 2 valves	Two valves at the inlet provides redundancy in case of problems.	Not applicable as can be seen/ established from visual inspection		
Surface structure for example Spillway						
Founded on soil	3	If founded wholly on rock then score 0; if on weathered rock on abutment score 1.	If founded on soil more vulnerable to concentrated leakage under foundation.	Not applicable as can be seen/ established from visual inspection		
Located on embankment	3	If on the abutment score 0. If any part of the structure is on the embankment score full marks.	Any leakage into/ along/ out of structure is more likely to lead to a failure of the embankment.	Not applicable as can be seen/ established from visual inspection		
Detailing of interface with embankment/ abutment poor (likely to lead to seepage/ low contact stresses at junction with watertight element)	2	From discussions with the supervising engineer. Consider the extent of the clay core and whether there is a cut off extended under the spillway channel too.	High vertical walls and/or separate backfill to excavation for the structure are weak points where concentrated leakage could occur, leading to internal erosion and failure.	2	2	0- back of walls battered to reduce risk of low contact stresses

				Fallback where no dam specific information is available: assume typical for date of construction		
Construction feature	**Max score**	**Guidance on scoring**	**Common potential failure mode(s)**	**18th century**	**19th century**	**Modern**
No downstream filter or filter zone at interface with embankment	1	Modern dams would have a filter around the structure, rear wall drainage along the back of the spillway walls, and so on. These designs would score 0.	Particle migration associated with concentrated leakage will not be stopped, allowing on-going erosion leading ultimately to failure.	1	1	0 – filter behind clay core extended to act as filter to any leakage along the contact between the structure and adjacent fill
Inadequate tail water	2	Design for < 25% of FRS Design Flood. Consider the size of the tail water structure. If the outfall is a long way from the embankment toe, then score 0 here.	Where the tail water is inadequate scour will occur, leading to undermining and ultimately collapse of the structure.	2	2	0

Table 8.19 Guidance on assigning embankment current condition score (Tier 2)

Indicator	Max score	Guidance on scoring	Common potential failure mode(s)	Suggested scores for various degrees of uncertainty		
				Unlikely	Not known, could be occurring	Likely
Seepage						
Large amount of uncontrolled seepage. that is not discharging to filtered drainage system	6	**a)** 10 times Seepage Index given in Charles et al. (1996, p. 7). **b)** The intention is that this is only scored if the quantities of seepage are higher than would normally be expected – thus the assessment should include an assessment of what the expected seepage would be and the score would be 0 for normal seepage. **c)** Where the local geology is such that significant seepage could be occurring undetected into permeable deposits in the valley floor (for example, cobbles/clean gravels), consider whether some score should be allocated for this uncertainty. c) where the seepage is emerging from non-erodible rock then the score may be reduced	Deterioration may lead to sudden failure; high flow increases risk of fines being transported.	0.3 – it is possible but unlikely	1.2 – unknown. For example, toe of dam submerged by downstream reservoir; or founded on very permeable deposits	3 – if it is unknown whether there is any seepage but there has been evidence of large volumes of seepage during the dams life and there has been no change in intrinsic condition such that this could be occurring again
Seepage increasing at same reservoir level	8	Maximum score when change of 20% on previous value Objective is to test whether there is a significant increase in seepage (after correction for any seasonal variation and influence of rainfall). Full marks would be given when the 20% change occurred over a year. If the change was over 3 years, score 4.	Changing conditions indicate deterioration.	0.4 where the published geological map indicates a granular alluvium or glacial in which seepage could be occurring	1.6	4 – if it is unknown whether there is any seepage, but there has been previous evidence of seepage linked to reservoir level during the dams life
Seepage carrying fines	10	Maximum score when Cloud of particles If there is no seepage, or the seepage is running clear, score 0. Note that very small quantities of suspended solids is not visible to the naked eye and can only be detected by settling out or turbidity meter	Loss of fines from the dam implies incipient failure.	0.5 (for example, no turbidity monitoring)	2	5
Increased pore pressures in/under downstream shoulder	6	Maximum score when Increase of 20% of reservoir head Score if pore pressures could increase with no visible seepage. This could occur where drains in the downstream shoulder block, or where there are high permeability bands in the foundation (as failure mode). Where there are no piezometers, consider likely piezometric behaviour. If behaviour is likely to be manifested by seepage/leakage (that is this is not a separate failure mode) then skip (that is score under seepage)	Failure mode likely to be different from deterioration involving increasing flows. that is there might be a pervious layer in the foundation which could lead to localised high pore pressures at the toe, which could then fail by uplift.	0.3	1.2 – Possible	3 – Likely but not certain
Animal burrows	1	Maximum score when extensive, that is, deep burrows (for example, badgers) or large numbers of shallower (rabbit)	Animal burrows may provide a preferential	Not applicable as can be seen in the field		

Indicator	Max score	Guidance on scoring	Common potential failure mode(s)	Suggested scores for various degrees of uncertainty		
				Unlikely	Not known, could be occurring	Likely
		burrows (say >20) in a position where there is a risk to the watertight element or other structural effect on the dam. If five or less shallow burrows, or moles score as 0.	seepage path through the watertight element, particularly under elevated reservoir level.			
Decaying tree roots	1	Extensive decaying tree roots in the vicinity of the crest/watertight element should score full marks.	Decayed tree roots can create a leakage path through the watertight element.	Not used	0.5 – Some decaying tree roots, which are not considered to be a threat to the ability to retain water in the reservoir	Not used
Deformation						
Settlement	4	Maximum score for acceleration with increase in gradient of > 50%, or: a) Absolute settlement > 3 times expected (Johnstone et al. 1999, p. 16). Acceleration change would be over a year after correction for reservoir drawdowns. b) Score 0.5 if 25% > expected; 1.0 if 50% > expected. c) Where the movement is longstanding and stable score ½ marks. d) Where settlement has been remediated and there is no new settlement score as zero (except if the cause of the movement was not fully understood score ½ marks that would be awarded based on observed settlement).	Changing conditions indicate deterioration.	0.2 – where no settlement monitoring and unlikely	0.8 – where no settlement monitoring and possible increase	2 – where no settlement monitoring and likely increase
Sinkholes, depressions, local settlement	10	Discussion with the Supervising Engineer and the Inspecting and Supervising Engineers reports. 4 marks for 0.1m, 6 marks for 0.3m deep, 6 marks for 1.0m deep. 'b' and 'c' as for settlement index. This is not measured by settlement pins unless installed to monitor a specific local feature	Local depression indicates internal erosion at depth within the dam.	Not applicable as can be seen in the field		
Slope movement (lateral deformation cracking)	4	Maximum score for Persistent crack of length > dam height Consider what could be causing any cracking (for example, desiccation), but these would normally vary in profile. Only mark cracks which relate to credible failure modes of the dam		Not applicable as can be seen in the field		

Table 8.20 Current condition of surface structure

Construction feature	Max score	Guidance on scoring	Common potential failure mode(s)	Suggested score for various degrees of uncertainty		
				Unlikely	Not known; could be occurring	Likely
Seepage						
Uncontrolled large quantity of seepage from cracks/ joints into/through structure, or emerging in vicinity of structure	4	Max score for 10 times Seepage Index given in Charles et al. (1996, p. 7) a) The intention is that this is only scored if the quantities of seepage are higher than would normally be expected – thus the assessment should include an assessment of what the expected seepage would be and the score would be 0 for normal seepage. b) Where the local geology is such that significant seepage could be occurring undetected into permeable deposits in the valley floor (for example cobbles/ clean gravels?), consider whether some score should be allocated for this uncertainty. c) Where the seepage is emerging from non-erodible rock then the score may be reduced.	Deterioration may lead to sudden failure; high flow increases risk of fines being transported.	0.2 – possible but unlikely	0.8 - E.g. end of structure submerged by downstream reservoir; or founded on deep very permeable deposits	2- e.g. there has been evidence of large volumes of seepage during the dams life and this could be occurring again
Seepage into/ from structure increasing at same reservoir level	6	Max score for change of 20% on previous value. Consider increase in seepage and whether it is linked to reservoir level or rainfall	Changing conditions indicate deterioration.	0.3	1.2	3
Seepage into /from structure carrying fines	8	Max score for cloud of particles If there is no seepage, or the seepage is running clear, score 0. Where the seepage is due to water entering from the spillway cute, downstream of the watertight element, score half.	Loss of fines from the dam implies incipient failure.	0.5	1.6	4
Deformation						
New cracks/ widening of existing cracks.	3	If there are no cracks, score 0. Where cracks has been remediated and there is no new cracks score as zero (except if the cause of the movement was not fully understood score half marks). Where the movement is longstanding and stable score half marks		Not applicable as can be seen in the field		

Construction feature	Max score	Guidance on scoring	Common potential failure mode(s)	Suggested score for various degrees of uncertainty		
				Unlikely	Not known; could be occurring	Likely
Deformation of embankment above/ adjacent to structure e.g. sinkholes	8	8 for 1m deep; 3 for 0.1m deep Discussion with the Supervising Engineer and the Inspecting and Supervising Engineers reports. If the depressions are not adjacent or local to the structure under consideration, score 0 here.	These are indicative of internal erosion and concentrated leaks along the contact between the structure and fill	Not applicable as can be seen in the field		
Other						
Scour at outlet from structure	2	Depth = 25% of width of structure Is there any evidence of erosion in the downstream structure/ channel? If the outlet to the structure is not close to the embankment and could not affect the stability of the dam, score 0	Scour can lead to structural collapse of the structure, and may also expose pervious foundation strata through which internal erosion could occur.	Not applicable as can be seen in the field		
Material deteriorating	3	Full marks where a) for reinforced concrete; the reinforcement is exposed and corroding for mass concrete/ brickwork; there has been a 20% loss of section. Is there any evidence that the material making up the structure is deteriorating, If there is definitely no signs, score 0.	Where the structural material is deteriorating, then this increases the vulnerability to structural collapse, or perforation which would allow a concentrated leakage which could erode fill material	Not applicable as can be seen in the field		

Table 8.21 Current condition of buried structure

Indicator	Max score	Guidance on scoring	Common potential failure mode(s)	Suggested score for various degrees of uncertainty		
				Unlikely	Not known; could be occurring	Likely
Uncontrolled large quantity of seepage from cracks/ joints into structure, or emerging in vicinity of structure	6	Max score for 10 times what would be normal for type of structure and dam height a) The intention is that this is only scored if the quantities of seepage are higher than would normally be expected – thus the assessment should include an assessment of what the expected seepage would be and the score would be 0 for normal seepage. b) Where the local geology is such that significant seepage could be occurring undetected into permeable deposits in the valley floor (for example, cobbles/ clean gravels?), consider whether some score should be allocated for this uncertainty.	Deterioration may lead directly to failure; high flow increases risk of fines being transported	0.3 – possible but unlikely	1.2 – unknown. For example end of structure submerged by downstream reservoir; or founded on deep very permeable deposits	3 – If it is unknown whether there is any seepage, but there has been evidence of large volumes of seepage during the dams life and this could be occurring again
Seepage into/ from structure increasing at same reservoir level	8	Max score for Increase of 20% on long term value Consider increase in seepage and whether it is linked to reservoir level or rainfall.	Changing conditions indicate deterioration.	0.5	1.6	4
Seepage into/ from structure carrying fines	10	Max score for cloud of particles If there is no seepage, or tunnel in competent rock, score 0.	Loss of fines from the dam implies incipient failure.	0.5 Unlikely; 0.5 if visually clear but no turbidity monitoring	2	5
Deformation						
New cracks/ widening of existing cracks,	4	Max score for 10mm width. If there are no cracks, score 0. Where cracks have been remediated and there are no new cracks score as zero (except if the cause of the movement was not fully understood score half marks). Where the movement is within the last 20 years and stable score half marks, where it dates from the original construction score zero.		Not applicable as can be seen/ established from visual inspection		
Deformation of embankment above/ adjacent to structure for example sinkholes	10	Max score for 10 for 1m deep; 4 for 0.1m deep This relates to local settlement related to internal erosion, not general embankment settlement (that is differential embankment to structure) for example sinkholes or other local settlement, or depression over line of outlet culvert. If the depressions are not adjacent or local to the structure under consideration, score 0 here (this should be picked up in Sheet 4.4, or in relation to the other structure).	These are indicative of internal erosion and concentrated leaks along the contact between the structure and fill.	Not applicable as can be seen/ established from visual inspection		
Other						

Indicator	Max score	Guidance on scoring	Common potential failure mode(s)	Suggested score for various degrees of uncertainty		
				Unlikely	Not known; could be occurring	Likely
Scour at outlet from structure	2	Max score for depth = 25% of width of structure Is there any evidence of erosion in the downstream structure/ channel? If the outlet to the structure is not close to the embankment and could not affect the stability of the dam, score 0.	Scour can lead to structural collapse of the structure, and may also expose pervious foundation strata through which internal erosion could occur.	Not applicable as can be seen/ established from visual inspection		
Material in contact with fill deteriorating	4	Full marks where for reinforced concrete; the reinforcement is exposed and corroding for mass concrete/ brickwork; there has been a 20% loss of section. This applies to the material forming the structural lining to the opening through the watertight element for example masonry, brick (or pipe if the pipe is laid directly within the fill). Is there any evidence that the material in contact with the fill/ natural ground is deteriorating? Score 0 if tunnel in competent rock.	Where the structural material is deteriorating, then this increases the vulnerability to structural collapse, or perforation which would allow a concentrated leakage which could erode fill material.	0.2	1 if structural support is a material which by virtue of its age and exposure condition may be deteriorating	2
Material behind structure deteriorating	4	Full marks where voids visible through joints; or other evidence of loss of positive contact between structure and adjacent fill. 25% of marks where clay extruding through brickwork This applies to the material immediate adjacent to the structure (for example backfill if laid in trench) is deteriorating. Consider the nature of the surrounding material.	If the soil around the outlet is deteriorating then increases the vulnerability to internal erosion.	0.2	0.8	2
Material not in contact with fill deteriorating	3	Max score for Severe corrosion Score when there is a risk that the pipework through the tunnel, and valves, may not be available for use in an emergency?	Serious corrosion of the pipework may lead to leak into the outlet, which could cause structural problems, and/or mean that the draw off is not available for use in an emergency.	Not applicable as can be seen/ established from visual inspection		

Table 8.22 Example event tree for concrete dam

Event tree analysis and likelihood of failure for Concrete dams

Key **Scenario 1 - Cell to be completed with verbal description of probability (cell below then autopopulated with probability)**

Scenario 1 - Justification for selected score (to be completed by user)

Scenario 2 - Step probability and Justification (to be completed by user)

Root cause	This project Branch/ Failure mode Descrip	Location	Phase as "Reclamation" Best Practice Training Manual (Chapter 24) for earth dams 1 Reservoir level	2 Initiation (Trigger/ root cause)	3 Continuation	4 Progression Stage 1	5 Progression Stage 2	6 Progression Stage 3	7 Unsuccessful detection/ intervention	8 Breach (break through of reservoir)
		Likelihood of	**Being full/ at elevated flood level**	**Some form of structural problem initiating**	**Likely to continue ? (no feature(s) to block progression towards failure)?**	**Describes the way in which the defect progresses from local defect to perforation of watertight element (major structural problem) with potential for catastrophic structural collapse**			**likely to progress? i.e. Unsuccessful in detection/ action being taken to prevent progression?**	**Defect leading to large scale structural collapse with full uncontrolled release of reservoir?**
External threats										
Floods	**Instability on lift joint**		High reservoir level (hydrostatic load) in extreme flood incl % blocked for SF on overturning (triangular pwp) 1. drops to unity (if below crest) 2. is value with WL at dam crest	Likelihood of flawed joint with no tensile strength, such that crack can initiate	No membrane or Drains which inhibits water under pressure penetrating crack	Horizontal crack propagates 50% of width	Critical crack length reached, section becomes potentially unstable in sliding or overturning SF< 1.0	Crack extends sufficiently to extend whole width (move ds 5mm)	Undetected, or even if detected unable to do anything	Movement progresses to state where blocks move apart width > 0.5H
	Mechanism					**C**	**F**	**G**		
	Parameter to assist in assess likelihood		*stability index (OTT)*	*Lift joint tensile strength*	*Detailing- upstream facing, alignment of concrete facing/hearting lift joint, water stops in CJ*	*Concrete Quality Index*	*Stability index (OTR)*	*vertical joint details*	*leakage/ settlement + drawdown capacity*	*Block width, 3D effects*
	Instability in foundation		High reservoir level (hydrostatic load) in extreme flood incl % blocked such that SF on overturning (triangular pwp) 1. drops to unity (if below crest) 2. is value with WL at dam crest	Crack initiates i.e. loss of bond on sides	No measures to inhibit water under pressure penetrating crack (Fine fill at heel, deep cut-off, foundation drain)	Horizontal crack propagates 50% of width	Critical crack length reached, section becomes potentially unstable in sliding or overturning SF< 1.0	Crack extends sufficiently to extend whole width (move ds 5mm)	Undetected, or even if detected unable to do anything	Movement progresses to state where blocks move apart width > 0.5H
	Mechanism					**C**	**F**	**G**		
	Parameter to assist in assess likelihood		*stability index (OTT)*		*Detailing*	*Foundation Quality Index*	*Stability index (OTR)*	*vertical joint details*	*leakage/ settlement + drawdown capacity*	*Block width, 3D effects*
Use to estimate likelihood of branch which is most likely to lead to failure (release of reservoir)										
		Likelihood (pick list)								
		Probability (Lookup)	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
		Justification for score								
		Likelihood (pick list)								
		Probability (Lookup)	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
		Justification for score								

Seismic	**Instability on lift joint**	Reservoir Level Normally taken as Top Water Level Assume virtually certain	seismic shaking causes SF (OTT) to reduce to 1.0 (or if > MCE then SF at MCE), and likelihood of flawed joint with no tensile strength	No membrane or Drains which inhibits water under pressure penetrating crack	Horiz Crack propagates 50% of width	Critical crack length reached, section becomes potentially unstable in sliding or overturning SF< 1.0	Crack extends sufficiently to extend whole width (move ds 5mm)	Virtually certain that unable to intervene	Sliding/ Overturning failure on lift joint, one or more blocks
	Mechanism (see Table 4.6)				C	F	G		
	Parameter to assist in assess likelihood		*stability index (OTT); Lift joint tensile strength*	*Detailing- upstream facing, alignment of concrete facing/hearting lift joint, water stops in CJ*	*Concrete Quality Index*	*Stability index (OTR) under TWL*	*vertical joint details*	*leakage/ settlement + drawdown capacity*	*Block width, 3D effects*
	Sliding at concrete/ foundation interface		seismic shaking causes SF (OTT) to reduce to 1.0 (or if > MCE, then SF at MCE), and likelihood of tensile crack/ loss of bond on sides	No measures to inhibit water under pressure penetrating crack (Fine fill at heel, deep cut-off, foundation drain)	Horiz Crack propagates 50% of width	Critical crack length reached, section becomes potentially unstable in sliding or overturning SF< 1.0	Crack extends sufficiently to extend whole width (move ds 5mm)	Virtually certain that unable to intervene	Sliding/ Overturning failure on lift joint, one or more blocks
	Mechanism (see Table 4.6)				C	F	G		
	Parameter to assist in assess likelihood		*stability index (OTT)*	*Detailing*	*Foundation Quality Index*	*Stability index (OTR)*	*vertical joint details*	*leakage/ settlement + drawdown capacity*	*Block width, 3D effects*
	Likelihood (pick list)								
	Probability (Lookup)	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
	Justification for score								
	Likelihood (pick list)								
	Probability (Lookup)	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
	Justification for score								
Internal threats									
Deterioration of structure foundation	**Differential settlement leading to cracking in body of dam**	Reservoir at TWL	Ongoing Seepage leads to stress changes in dam foundation, and local area of reduced compression stress, which leads to differential settlement	No feature to prevent deterioration e.g. Drains too far apart/ insufficient capacity to intercept flow	Quality of foundation construction such that crack progresses	Concrete gravity section becomes potentially unstable in sliding or overturning SF< 1.0	watertight element perforated + wall moves/ vertical joints open by say 5mm	Undetected, or even if detected unable to do anything	Blocks move bodily downstream creating opening > 50% of dam height
	Mechanism (see Table 4.6)				D	F	G		
	Parameter to assist in assess likelihood					*Stability index (OTR) at TWL*	*vertical joint details*	*Surveillance + drawdown capacity*	*Block width, 3D effects*
	Internal erosion of soil foundation	Reservoir at TWL	Head > critical gradient	No feature to prevent deterioration e.g. Downstream filter	Quality of foundation construction such that internal erosion likely to continue	Erosion tunnel migrates upstream, Break through into reservoir	watertight element perforated + wall moves/ vertical joints open by say 5mm	#N/A	Hole enlarges, undermines weir which collapses
	Mechanism (see Table 4.6)				D	E	G		
	Parameter to assist in assess likelihood						*vertical joint details*	*Surveillance + drawdown capacity*	*Block width, 3D effects*
	Likelihood (pick list)								
	Probability (Lookup)	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
	Justification for score								
	Likelihood (pick list)								
	Probability (Lookup)	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
	Justification for score								

Table 8.23 Example event tree for service reservoir supported with structural fill

Event tree analysis and likelihood of failure for Service reservoris

This project			**Phase as "Reclamation" Best Practice Training Manual (Chapter 24) for earth dams**							
Root cause			1	2	3	4	5	6	7	8
	Descrip	**Location**	**Reservoir level**	**Initiation (Trigger/ root cause)**	**Continuation**	**Progression Stage 1**	**Progression Stage 2**	**Progression Stage 3**	**Unsuccessful detection/ intervention**	**Breach (break through of reservoir)**
		Likelihood of	**Being full/ at elevated flood level**	**Some form of structural problem initiating**	**Likely to continue ? (no feature(s) to block progression towards failure)?**	**Describes the way in which the defect progresses from local defect to perforation of watertight element (major structural problem) with potential for catastrophic structural collapse**			**likely to progress? i.e. Unsuccessful in detection/ action being taken to prevent progression?**	**Defect leading to large scale structural collapse with full uncontrolled release of reservoir?**
External threats										
Excessive inflow	**Overtop; leads to failure of supporting fill**		Reservoir rises to top of perimeter wall (alarm fails or is ignored)	Reservoir continues to rise and reservoir roof lifts off, so can spill over perimeter	Release over wall is concentrated over 1 or 2 bays so unit discharge is > 0.02 m3/s/m	Overtopping flow penetrates interface between wall and fill, and/or scours surface (damage embankment)	Stability failure of perimeter embankment	Section potentially unstable, watertight element perforated + wall moves/ vertical joints open by say 25mm	Not detected, and even if detected too late to visit WTW to close valves	Perimeter wall units move / separate leading to gap at least equal to 50% of dam height
	Parameter to assist ranking		*Overflow capacity*	*As Step 1*		*Mechanism A*	*Slope ranking Index*	*Stability ranking Index*		
	Instability in body of perimeter wall			Increased loading leads to tensile stress at wall foundation - cracks initiates	No measures to inhibit water under pressure penetrating crack (u-s membrane, internal drain)	Quality of construction such that crack can propagate	Concrete gravity section becomes potentially unstable in sliding or overturning SF< 1.0	watertight element perforated + wall moves/ vertical joints open by say 25mm	Not detected, and even if detected too late to visit WTW to close valves	Perimeter wall units move / separate displacing perimeter fill, leading to gap at least equal to 50% of dam height
	Parameter to assist ranking			*Stability Index*		*Concrete quality Index*	*Stability ranking Index*			
	Instability of foundation			Increased loading leads to tensile stress at wall foundation - cracks initiates	No measures to inhibit water under pressure penetrating crack (u-s membrane, internal drain)	Quality of construction such that crack can propagate/ foundation deforms	Concrete gravity section becomes potentially unstable in sliding or overturning SF< 1.0	watertight element perforated + wall moves/ vertical joints open by say 25mm	Not detected, and even if detected too late to visit WTW to close valves	Perimeter wall units move / separate displacing perimeter fill, leading to gap at least equal to 50% of dam height
	Parameter to assist ranking			*Stability Index*		*Foundation quality Index*	*Stability Ranking Index*			
			Estimate likelihood of branch which is most likely to lead to failure (release of reservoir)							
		Likelihood (pick list)								
		Probability (Lookup)	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A

Seismic	**Stability failure of external structural fill**	Reservoir just below Hi alarm	seismic shaking causes instability in structural fill	No feature to prevent instability propagating e.g. wall dependent on fill	Fill becomes potentially unstable in sliding or overturning SF< 1.0	Concrete gravity section becomes potentially unstable in sliding or overturning SF< 1.0	watertight element perforated + wall moves/ vertical joints open by say 25mm	Not detected, and even if detected too late to visit WTW to close valves	Perimeter wall units move / separate leading to gap at least equal to 50% of dam height
	Parameter to assist ranking		Stability Index		Slope Index	Stability index	Vertical joint details		
	sliding in body of gravity dam	Reservoir just below Hi alarm	seismic shaking causes cracking/ tensile cracks in body of dam	No feature to prevent crack propagating e.g. no waterbars in lift joints	Quality of concrete construction such that crack likely to propagate	Concrete gravity section becomes potentially unstable in sliding or overturning SF< 1.0	watertight element perforated + wall moves/ vertical joints open by say 25mm	Not detected, and even if detected too late to visit WTW to close valves	Perimeter wall units move / separate displacing perimeter fill, leading to gap at least equal to 50% of dam height
	Parameter to assist ranking	*Observed weekly max/min WL over last ten years*	*Critical EQ*		*Concrete quality Index*	*Stability Ranking Index*			
	Likelihood (pick list)								
	Probability (Lookup)	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Internal threats									
Deterioration of material forming body of structure	Leakage through floor/wall joint	Reservoir just below Hi alarm	Deterioration of waterstop or movement at joints leads to significant leakage under perimeter wall, which initiates internal erosion	No filter, or deep perimeter cut-off which would prevent migration of fines; Leakage sufficient flow to erode fines	Quality of foundation construction such that internal erosion likely to continue	Cavity migrate under perimeter wall foundation	watertight element perforated + wall moves/ vertical joints open by say 25mm	Not detected, and even if detected too late to visit WTW to close valves	Perimeter wall units move / separate displacing perimeter fill, leading to gap at least equal to 50% of dam height
	Parameter to assist ranking								
	Likelihood (pick list)								
	Probability (Lookup)	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Deterioration of structure foundation		Reservoir just below Hi alarm	Deterioration of structure foundation leading to cracking of structure	No feature to prevent fdn deterioration continuing	Quality of foundation construction such that crack progresses	Concrete gravity section becomes potentially unstable in sliding or overturning SF< 1.0	watertight element perforated + wall moves/ vertical joints open by say 25mm	Not detected, and even if detected too late to visit WTW to close valves	Perimeter wall units move / separate displacing perimeter fill, leading to gap at least equal to 50% of dam height
	Parameter to assist ranking								
	Likelihood (pick list)								
	Probability (Lookup)	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Deterioration of pipework		Reservoir just below Hi alarm	pipe fractures (inlet/ outlet/ cross connection) in body of supporting fill. Could be deterioration of pipe or diff settlement	No pipe burst protection system	saturation of supporting fill	successive slip of embankment	Section potentially unstable, watertight element perforated + wall moves/ vertical joints open by say 25mm	Not detected, and even if detected too late to visit WTW to close valves	perimeter wall fails in vicinity of pipes (but remote from valve house)
	Parameter to assist ranking						*Stability ranking Index*		
	Likelihood (pick list)								
	Probability (Lookup)	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A

Table 8.24 Quantitative descriptor mapping scheme for event trees for gravity dams

Descriptor assigned by workshop	Probability – Note 1 (look-up function)	Comment
Very likely/Virtually certain	0.99	Chapter 8 of BPTM (Reclamation 2009) except that 'Virtually certain (0.999)' not used as no practical difference from 0.99.
Likely	0.9	
Neutral	0.5	
Unlikely	0.1	
Very Unlikely	0.05	Additional descriptor, as used on Loyne dam (Mason 2010)
Highly unlikely	0.01	
Virtually Impossible	0.001	
Notes	[1] For initiation this is annual chance, for other phases it is chance of proceeding to next step. [2] For initiation due to external threats - normally consider dam critical, or other extreme load– as Figure 8.3. [3] The above may be overwritten (a) when the initiating probability derived from assessment of floods/ earthquake is less than 1 in 1,000; (b) Using slope and stability index factors.	

Table 8.25 Likelihood of service reservoir water level reaching underside of roof

	Factor	Typical likelihood in any year	Factors increasing/ reducing likelihood
1	Likelihood of physical failure of instrumentation (level sensors/ alarms)	1 in 100 for failure	
2	Lack of response to alarm	Base on number of times reach HIHI alarm each year	
3	Magnitude of inflows	Assume 100% of water treatment works (WTW) output	
4	Demand	Overnight demand small. At this screening stage assume zero.	
5	Capacity of any overflow provision	Where not available, approximate estimate can be obtained using smallest diameter of pipe between service reservoir and watercourse, and velocity of 4m/s.	Magnitude of inflow > overflow capacity for example if works increased in capacity after service reservoir built. Where overflow capacity > inflow, score as virtually impossible (0.001).
6	Risk of blockage of overflow, or overflow pipe/ culvert	Risk of blockage small (covered treated water)	

Table 8.26 Likelihood of localised section of service reservoir roof being displaced, providing concentrated overflow route

	Factor	Comment	Typical likelihood in any year
1	Form of connection between roof and walls	Simply supported roof beams on RC structures Barrel vaulted brick roofs Tied connections	Likely (0.9) Neutral (0.5) Unlikely (0.1)
2	Depth of roof/ cover	Dead weight of roof and covering topsoil	Use Table 8.25 to infer build-up of static water pressure required to lift roof.
3	Chance of detection that it is overflowing	Depends on frequency of surveillance.	Normally neglect (highly unlikely)
4	Ventilator detailing	May prevent release of air/ contribute to roof failure. Historically may have provided additional overflow; but now often subject to security restrictions which may limit ability to act as secondary overflows.	

Table 8.27 Concrete quality index (likelihood that crack in lift joint in gravity dam progresses)

Likelihood of Phase 4 progressing (for example, crack propagating)		Date of construction (Dolen 2011)	Quality of concrete/ masonry construction	Current condition/ performance
Likely	0.9		Lift joint already cracked due to earthquake, or de-bonded because of poor construction/ deterioration	Leaking on lift joints No bond in cores
Neutral	0.5	Pre 1930	Unknown/ poor	
Unlikely	0.1			
Very unlikely	0.05	Post 1930	Good	
Highly unlikely	0.01			
Virtually impossible	0.001		Evidence that engineering included positive measures to ensure high bond	
Other factors that could be included: inclination of lift joints to horizontal				

Table 8.28 Likelihood of foundation deterioration of gravity structures

	Factor	Comment	Typical likelihood in any year	
			Initiation (Phase 2)	**Progression phases 4–6, Mechanisms D**
	Construction records	Log of formation encountered at founding level, treatment of soft spots	• Where no records assume highly unlikely, that is,1 in 100. • Where good quality records assume virtually impossible, that is, 1 in 1,000.	• Neutral • Good – assessment based on below
Factors increasing/ reducing likelihood				
1	Depth of foundation below OGL		Higher for shallow, unless positive evidence that competent strata at shallow depth	
2	Solid geology	Strata type/ strength, depth of weathering,		
3	Structural geology	Faults, dip	Higher where faults which are likely to create weak zones.	
4	Performance since original construction	Settlement, seepage		
5	Presence of soil Highly weathered rock	More likely with service reservoir where crossfall at OGL, so one corner at OGL		

OGL = ordinary ground level.

Table 8.29 Factors affecting likelihood of Mechanism G (structural movement sufficient to perforate watertight element

Feature	Typical likelihood of crack
Vertical joints in wall, and reinforcement across joints	Likely (0.9) if unreinforced movement joints No joints – Neutral (0.5) Virtually impossible (0.001) if continuous reinforcement as would have to shear steel/ fail in tension
Factors increasing/ reducing likelihood	
Two-dimensional effects – is there a potentially unstable length >H?	Less likely where section with low safety factor is limited in length.
Curved alignment in plan (arching action)	Less likely where geometry of arch is such that dam could to fail even with 10mm crack (that is, lock up on arch).

Notes: For example >25mm in service reservoir, 10mm in concrete/masonry dam.

Table 8.30 Likelihood of unsuccessful detection/ intervention (Phase 7)

	Threat	Typical likelihood
	Base case	‘Very likely’ (0.99) to progress (be unsuccessful at detection and intervention) where visual inspection is only once a week ‘Neutral’ (0.5) likelihood of progressing (be unsuccessful at detection and intervention) where visual inspection is three times/ week NB: Although some service reservoirs are next to water treatment works (WTW) that are manned during normal working hours, WTW staff generally don’t have to pass around the whole perimeter on their way to/from work.
‘	**Factors increasing/ reducing likelihood**	
	Frequency of surveillance	Reduce where increased frequency of surveillance or manned WTW and staff pass whole length of perimeter.
21	Deterioration of dam body leading to saturation/ settlement of perimeter bank.	Site staff likely to notice on weekly surveillance visits so likelihood of progress very unlikely (0.05).
22	Deterioration of dam foundation	Site staff likely to notice so likelihood of progress very unlikely (0.05).
23	Deterioration of pipework	Fracturing of a pipe in the fill is likely to affect either the WTW process or supply pressures such that unlikely to progress (not be detected and action taken to prevent failure of reservoir).

Table 8.31 Factors governing likelihood of full breach of gravity dam supported by external fill

Feature	Comment	Typical likelihood of breach (for floor at OGL, unreinforced vertical movement joints)
Perimeter fill	Where has not previously failed resistance will build up to Kp as wall moves. Although could be taken as the likelihood of Mechanisms A and B, suggest the approach here is used as failure is initiated in a different way.	Threat 1.1 – Fill has already been eroded due to overtopping (gully) – Unlikely (0.1) Threat 2.1 – Although perimeter bank failed, no energy to remove so likely to be largely in place – Very unlikely (0.05) Fill has not failed (Threats 1.2, 1.3, 2.2, 2.3, 4) – Highly unlikely (0.01) unless fill may saturate and fail (0.05)
Factors increasing/ reducing likelihood		
Embedment	Increasing embedment likely to inhibit full release	Neutral where no embedment. Highly unlikely (0.01) for 2m embedment
Size of reservoir/ rate of drain down of reservoir	Net outflow (inflow less leakage through defect) may be significant such that head insufficient to complete breach	Plane impact – assume neutral (0.5) where continuous wall with no joints that is fuselage punctures walls but does not displace blocks to create a large opening. Refine assessment by considering volume of fill to be eroded, and assume volume of water required = say 10 times volume of fill.
Structural continuity along wall	Full continuity of steel/ dowel bars between reinforced concrete panels	Full continuity of steel – Virtually impossible (0.01)
	Shear keys between mass concrete	For Threat 1.1 (overtopping) will inhibit localised failure at point of overtopping–-Highly unlikely (0.01)
	3D effects	
Type of perimeter wall	Where just inner lining to perimeter bank then more likely	Neutral on basis that although still have to erode embankment to full depth over width of 50% height, the perimeter wall is thin and may fail by toppling into reservoir

Table 8.32 Factors governing likelihood of full breach of freestanding gravity dam body

Issue		Comment
Base case – 2D wall with vertical joints and no shear keys		Likely (0.9)
Factors increasing/ reducing likelihood		
Shear keys	Provides interlock between blocks, such that single block unlikely to fail on its own, and minimum failure likely to involve say three blocks.	
Spacing of vertical joints, relative to dam height	Provide release surfaces for catastrophic failure (where no joints then vertical cracks have to develop for full height of dam).	
Embedment of foundation below original ground level	Will reduce risk of sliding, particularly if partially embedded into weathered rock.	Say very unlikely (1 in 20 chance)
Landscaping/ structural fill downstream	Although considerations of stress/ strain mean that will not contribute significantly to preventing shear cracking of dam (that is, maximum pressure is at-rest (k_o), once significant lateral displacement occurs the lateral pressure must increase to full passive (k_p) if the soil is to be displaced such that the concrete blocks are released.	
Plan alignment	Dams which are curved in plan, with the middle further upstream than the downstream will have some arching effect, such that the likelihood of bodily movement of blocks downstream is reduced.	

Figure 8.12 Sliding stability index for gravity structures – static failure

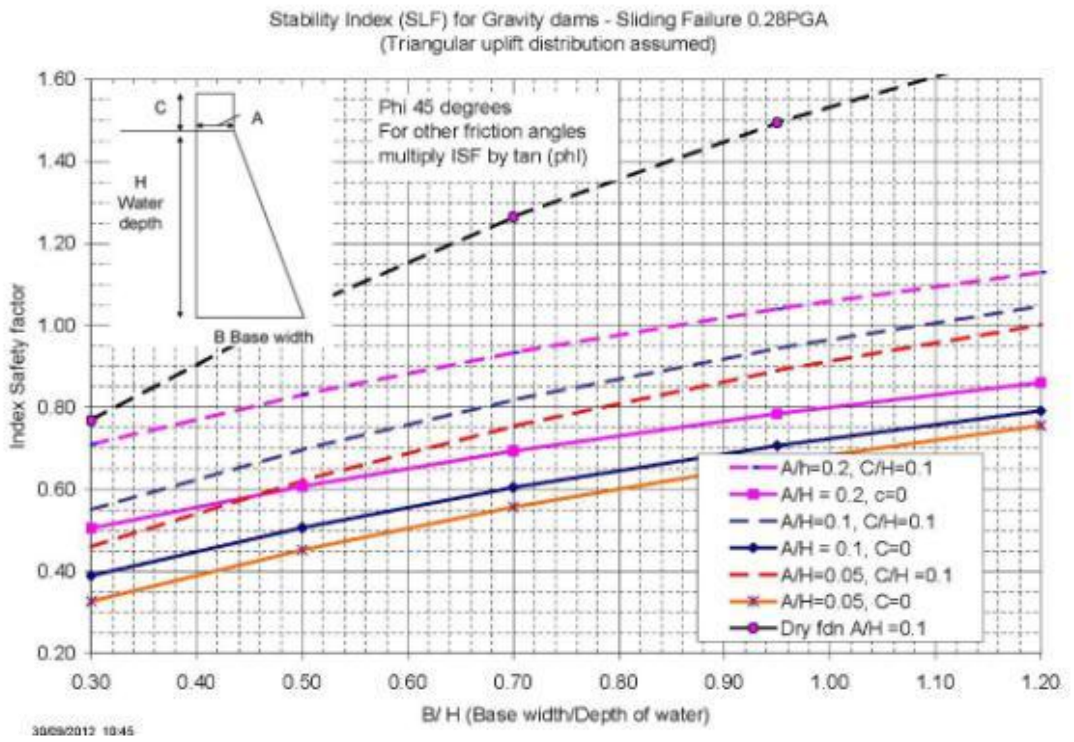


Figure 8.13 Sliding stability index for gravity structures – failure under earthquake

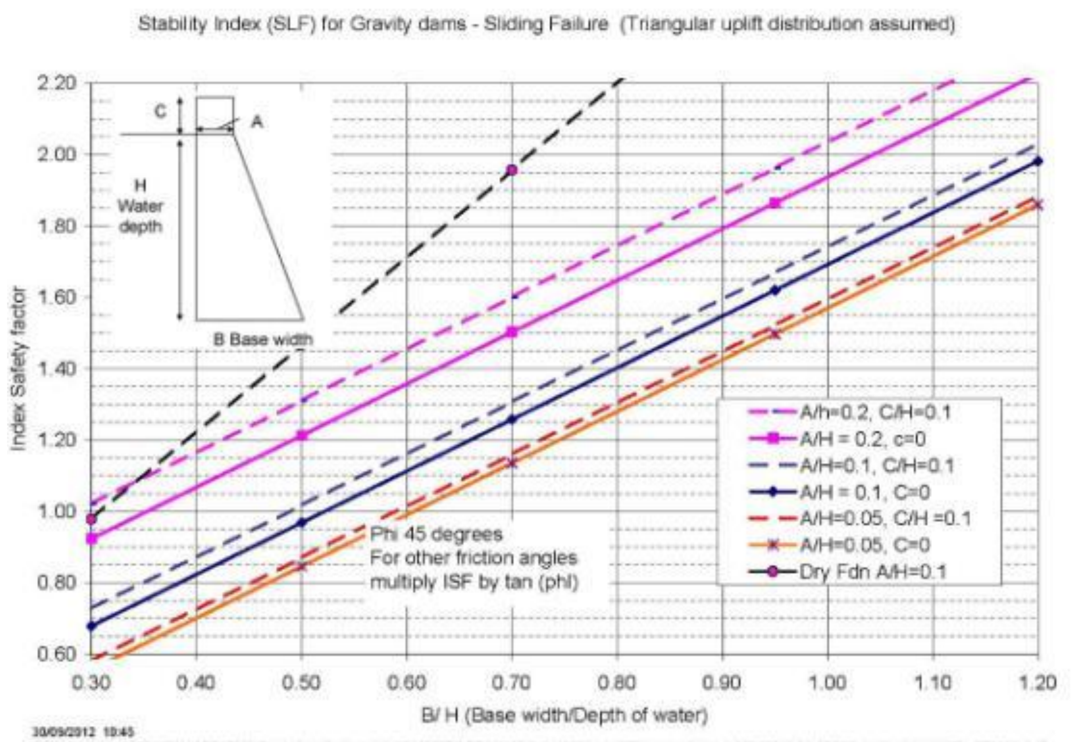


Figure 8.14 Overturning stability index for gravity structures – static failure with triangular uplift

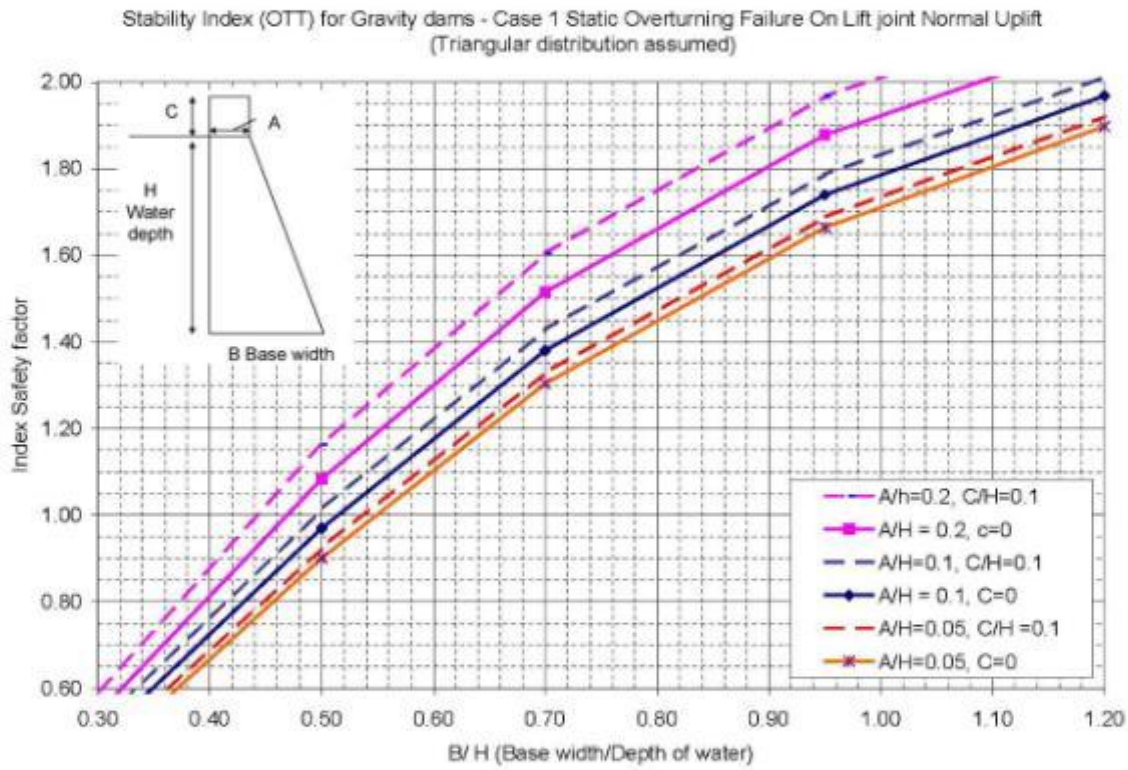


Figure 8.15 Overturning stability index for gravity structures – static failure with rectangular uplift

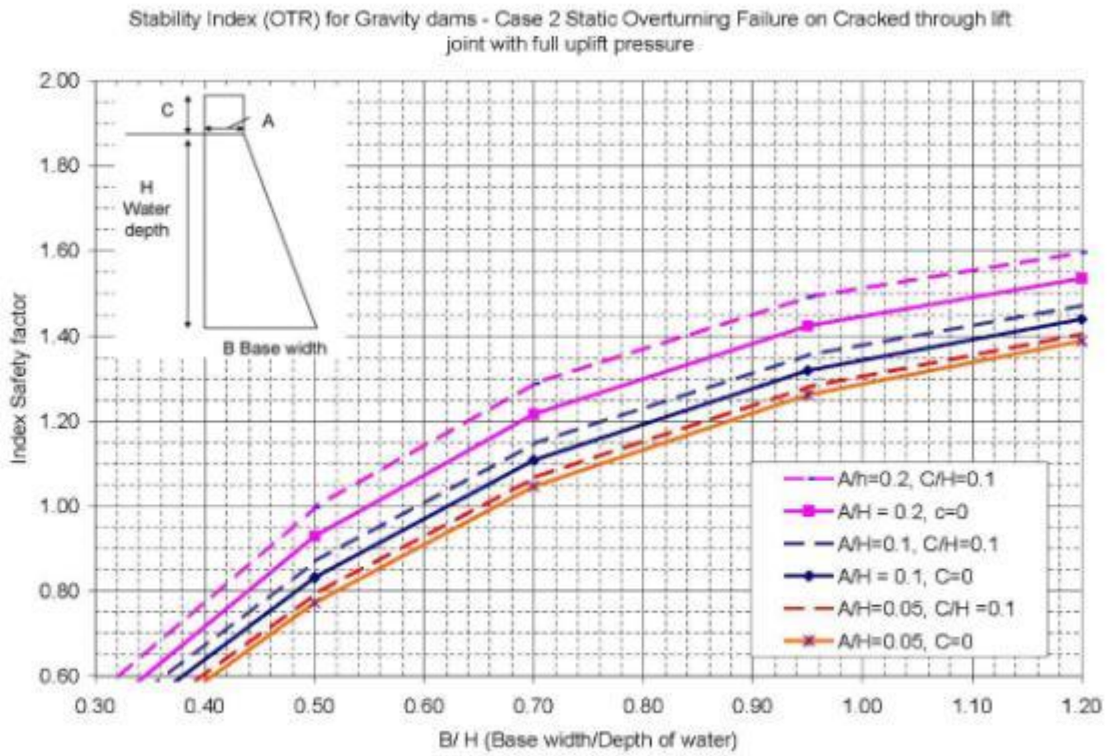
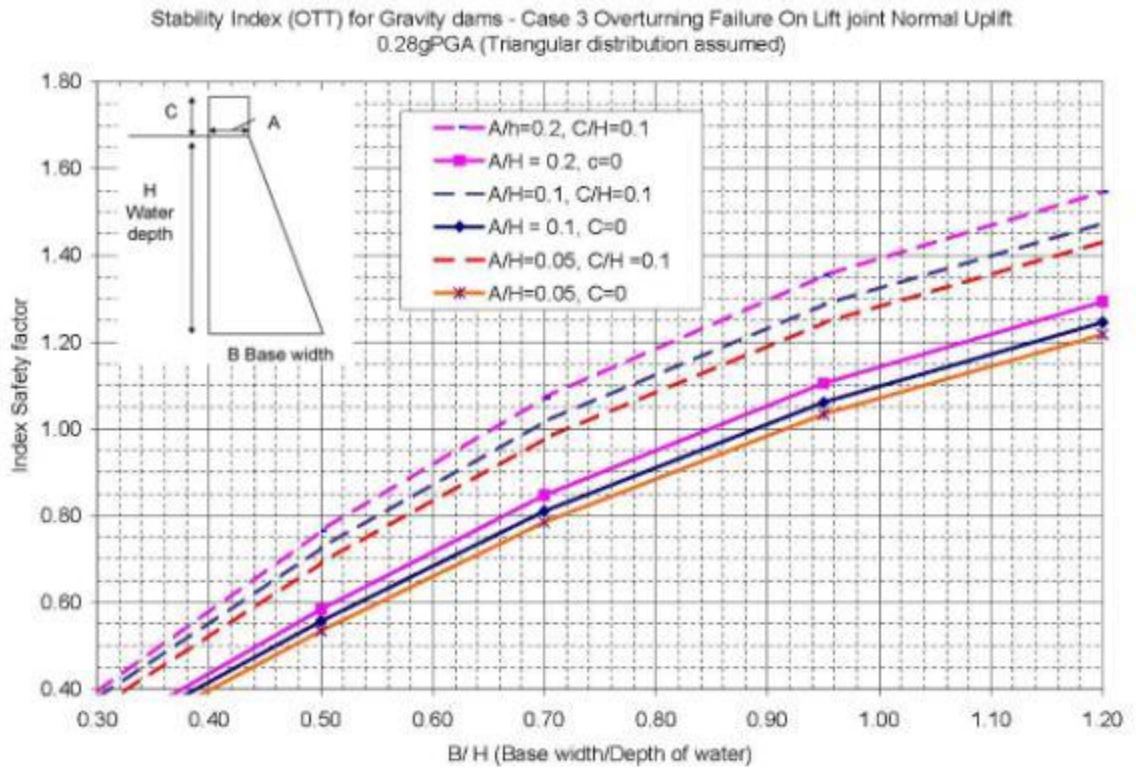


Figure 8.16 Overturning stability index for gravity structures – 0.28 earthquake with triangular uplift



9 Tier 2 – Steps 2c–f Dam breach, flood routing, consequences of failure and risk

Step 2	Risk Analysis	Step 2c	Dambreak and flood routing
		Step 2d	Consequence analysis
		Step 2e	Determine level of risk
		Step 2f	Review outputs
		Optional	Estimate range of uncertainty

9.1 Step 2c – Dam breach and flood routing

9.1.1 Dam breach

The speed and depth of flooding downstream following a dam failure depends on the extent of the dam breach and the speed with which the water is released. However determining this requires detailed dam breach modelling. A conservative assumption is to assume an instant dam failure of full dam height.

Two main approaches apply for estimating the potential inundation area, either by using existing dam break maps where available (of an appropriate standard), or by undertaking a new dam breach assessment and flood routing using the simplified methods presented below and taken from Hewlett et al. (2000) and refined in *An Interim Guide to Quantitative Risk Assessment for UK Reservoirs* (Brown and Gosden 2004).

Embankment breaching

For embankment breach prediction, use the Froehlich (1995) peak discharge equation combined with hydrograph shape estimation using some simple rules. Where appropriate, the prediction may be refined using soil erodibility and reservoir area data to guide on the likely nature of the flood hydrograph (Morris 2012).

The recommended discharge equation is based on observed data records from breach events and offers a peak discharge based on the best fit to observed data (Froehlich 1995).

To predict the peak discharge possible from a breach in an embankment dam apply:

$$Q_p = 0.607V^{0.295} H^{1.24}$$

where:

Q_p peak discharge (m^3/s)

H height of reservoir water level above the flood plain at time of breach (m)

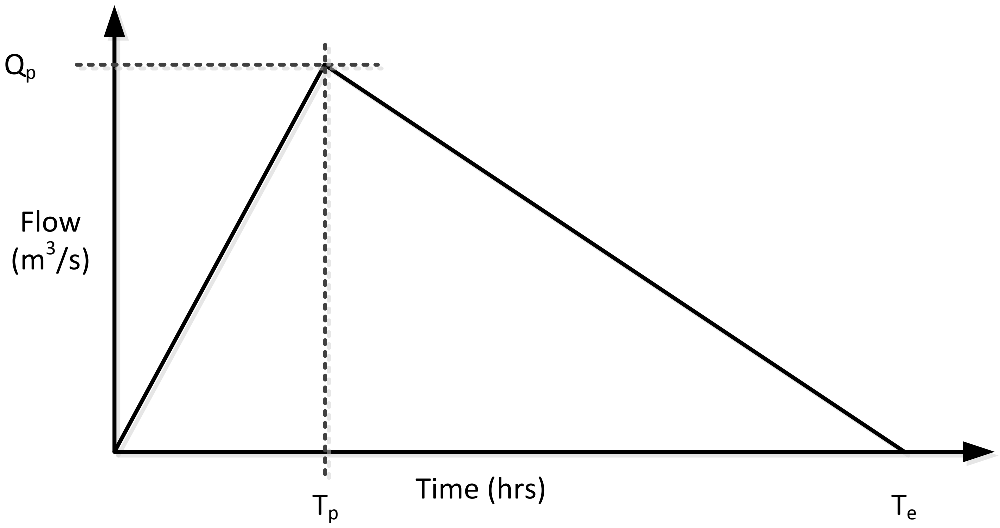
V storage volume of reservoir below level H (m^3)

To predict the time of failure to peak discharge, apply:

$$\text{Time to peak discharge, } T_p \text{ (s)} = 120\ H$$

where H is the height of peak reservoir water level above the base of the dam (m).

To estimate the shape of the flood hydrograph an approximation is to assume a triangular profile as shown below:

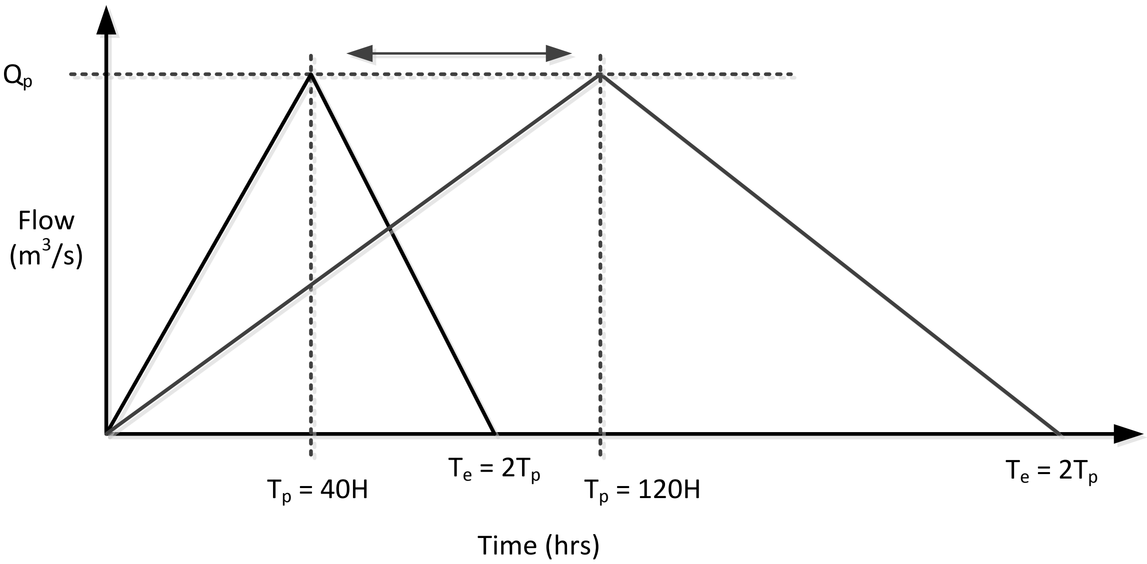


This hydrograph will be used later in predicting flood levels downstream of the dam. Given the volume of water stored in the reservoir, an estimate of the flood hydrograph shape may be made where both Q_p and T_p have been calculated from the equations above. T_e may be calculated by ensuring that the volume under the hydrograph matches the reservoir volume, V, that is:

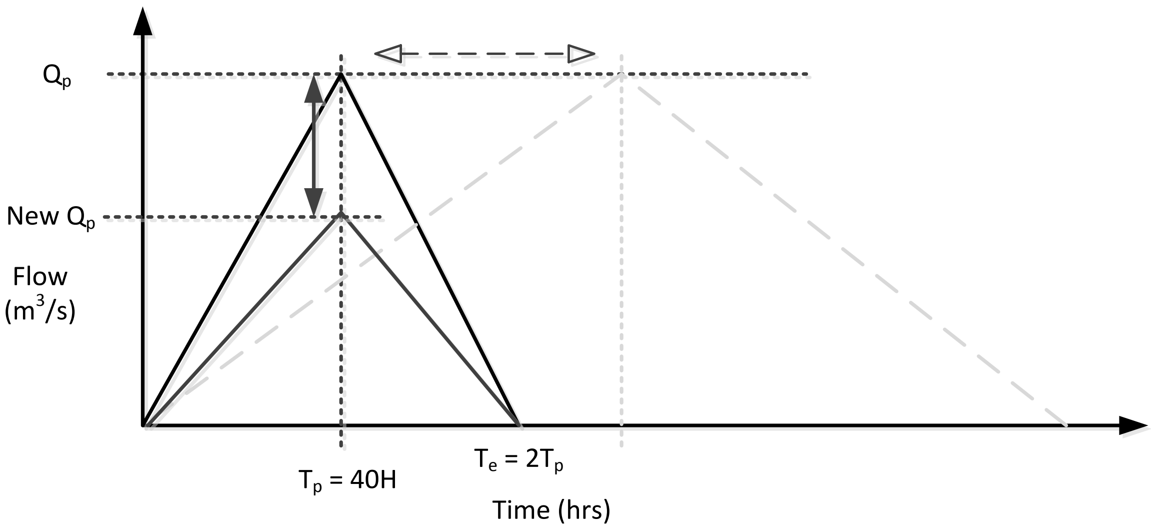
$$V = ½(Q_p \times T_p) + ½(Q_p \times (T_e - T_p))$$

Under some conditions, for example, where a dam stores a relatively small volume of water but has a relatively high embankment, the calculation of a flood hydrograph shape similar to that shown above will not be possible. When the value of T_e is less than twice the value of T_p, or the value of T_e cannot be calculated, then keeping the value of Q_p constant (as previously calculated) reduce T_p, while at the same time keeping T_e at a value of $2T_p$. Do not reduce T_p to less than 40H. Under these conditions the flood hydrograph will therefore be symmetrical, with a peak value of Q_p and a peak time, T_p, of somewhere between 40H and 120H.

If a solution can be found at this stage, this is the correct flood hydrograph to use in later calculations.



If the volume of water represented by the smallest hydrograph (that is, the one with T_p = 40H) is still greater than the stored reservoir volume, then keeping T_p as 40H (and hence T_e as 80i), reduce the magnitude of Q_p until a volume balance between the flood hydrograph and the water stored in the reservoir can be achieved. This then represents the flood hydrograph to be used in later calculations.



Breaching of concrete dam types

See Table 19.2 for simplified methods specific to different types of dam.

Peak discharge from a failed concrete dam may be best estimated using:

$$Q_p = cLH^{1.5}$$

where:

Q_p peak discharge (m^3/s)

H height of peak reservoir water level above the downstream river valley (m)

L length of the dam across the valley (at the selected reservoir water level) (m)

c coefficient given by $c = 0.9R^{0.28}$

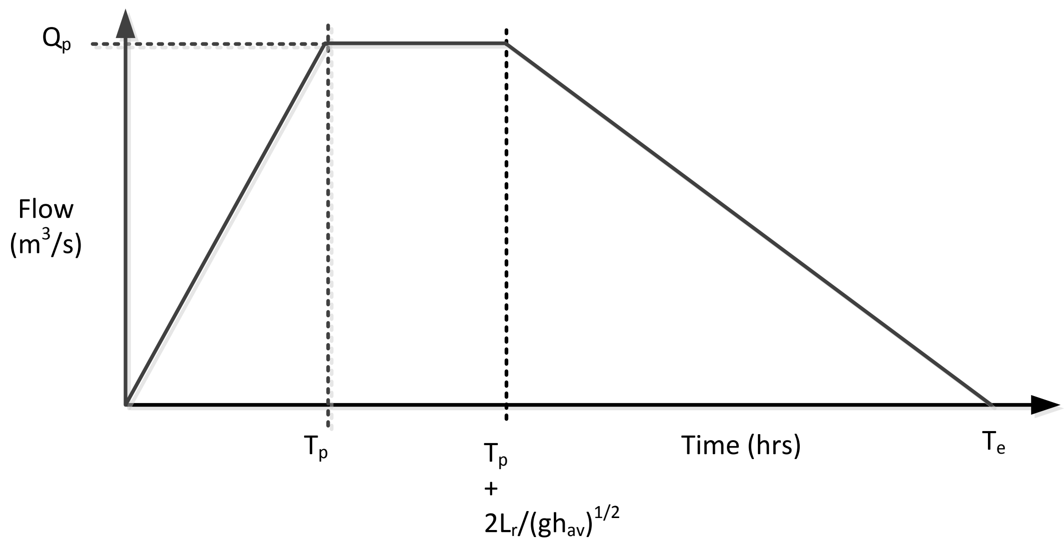
where R is the ratio between the breach area and total dam face area.

Note that the dam face area and breach area should relate only to areas bounded by the assumed reservoir water level. Therefore, if a dam is 50m high and has an assumed failure water level of 45m, then the width of dam (L), the breach area and dam face area should all be calculated using the 45m H value and not the 50m construction value. The 50m H value would only be used if it assumed that the reservoir was full to overtopping at the point of failure.

To determine R a decision has to be taken as to the probable breach size. Likely breach scenarios may be based upon site-specific construction details. Where the dam construction is based on clearly defined units, for example, it would be appropriate to consider failure of individual or multiple units. Where it is not possible to determine a likely failure mode from the site-specific design, the following guidance is given (where the breach is assumed to have vertical sides):

Type	**Breach width**	**Formation time T_p**
Arch	0.9L	30 s
Multiple arch and buttress	0.9L	30 s
Gravity arch	0.65L	30 s
Gravity	0.5L	720 s

Given the volume of water stored in the reservoir, an estimate of the flood hydrograph shape may now be made. When 'instantaneous' failure occurs, a shock wave travels up and back down the reservoir. During this period, the discharge from the dam remains approximately constant. The shape of the flood hydrograph approximates to:



where:

Q_p has been calculated from the earlier equation.

T_p may be taken from the table above.

H_{av} is the average depth of water along the reservoir.

L_r is the length of the main body of the reservoir (that is dam to upstream end).

T_e may be calculated by ensuring that the volume under the hydrograph matches the reservoir volume, V, that is:

$$V = ½(Q_p \times T_p) + Q_p \text{ x } 2L_r/\ (gH_{av})½ + ½(Q_p \times (T_e – T_{p}\_2L_r/\ (gH_{av})½)$$

This hydrograph will be used later in predicting flood levels downstream of the dam.

Where failure is not instantaneous (as with a gravity dam where failure time is taken as 720 s), calculation of the flood hydrograph is undertaken in a similar way to that outlined for the embankment failure (see above). However, the peak discharge, Q_p, is calculated using the equation for concrete dams given above.

Breaching of service reservoirs

Where a service reservoir has been constructed using a combination of concrete, masonry and brickwork, the prediction of breach formation, discharge and location is difficult. It is likely that peak discharge will fall somewhere between that calculated using concrete and the embankment dam equations. An appropriate value should be selected after due consideration of the structure design and the potential discharge, assuming it to comprise either an embankment or concrete structure.

Note that the two types of failure create floods with differing characteristics. An embankment failure is relatively slow and produces a longer flood hydrograph, but with a potentially lower and/or less prolonged peak discharge. A concrete failure, however, creates an almost instantaneous peak discharge, leading to a relatively short but high-intensity flood hydrograph. This type of failure is therefore likely to create the worst flood conditions downstream.

Breach of reservoir in cascade

When considering the potential for the cascade failure of dams, consider the combined cascade volume of water that might be retained by the lower dam, along with a retained water level above the dam crest level necessary to trigger breach. The combined retained volume and height may then be used within the Froehlich equation to provide an estimate of the potential cascade outflow from the dam.

9.1.2 Flood routing

The methodology requires division of the inundation area into zones of similar valley section and similar impact (for example rural, urban and so on), which should have been carried out as part of Step 1b. For each zone it will be necessary to define a typical valley width and side slopes, necessary to calculate an approximate value of flow depth and velocity to support estimation of likely loss of life, property damage and other impacts.

The technique is to apply Manning's equation using an estimate of the peak discharge for the given location:

$$\mathbf{Q_p = (A^{5/3}S_o^{1/2})/(nP^{2/3})}$$

where:

Q_p peak discharge at the calculation point (m^3/s)

A flow cross-sectional area (m^2)

S_o slope along the river valley

n Manning's roughness coefficient

P wetted perimeter of valley section (m)

Q_p is initially estimated at the dam using the breach equation. The magnitude of Q_p will reduce as the flood wave travels down the valley, due to attenuation. Therefore, to estimate the water level at the intersection between each valley zone, a new value of Q_p should be calculated. This may be done using the techniques outlined below:

$$Q_p(x) = Q_p(0)\exp[-x/L_a]$$

where:

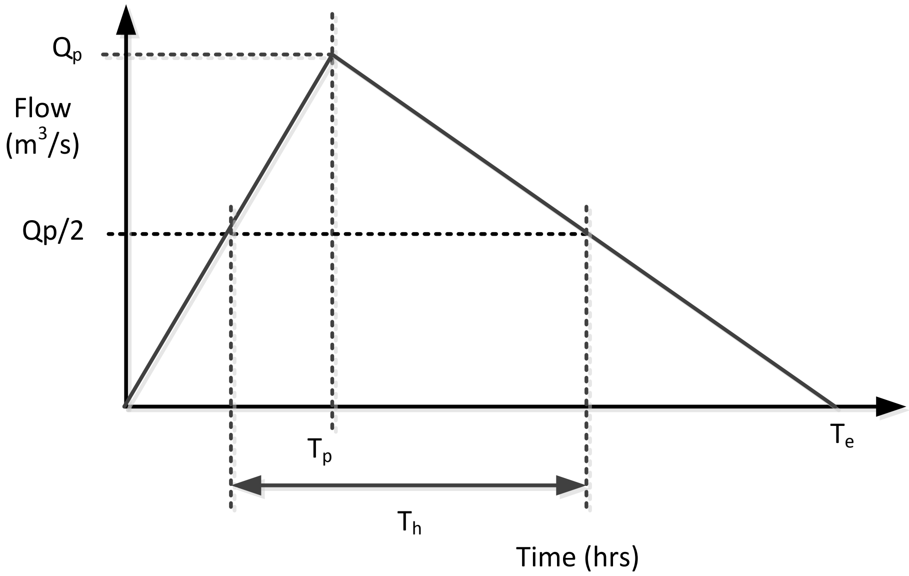
$Q_p(x)$	discharge at a location Xm downstream of $Q_p(0)$ location (m^3/s)
$Q_p(0)$	discharge calculated at upstream location (m^3/s)
x	distance between zone intersections (that is, length of the zone across which the calculation is being made, not the chainage downstream of the dam) (m)

and:

$$L_a - k\, B^{-0.2} S_0^{1.9} n^{-1.8} Q_p(0)^{0.2} T_h^{2}$$

where:

k	factor, with suggested value given in Table 9.1
B	estimated surface width of valley at estimated water depth (m)
S_0	valley slope
n	Manning's n
$Q_p(0)$	discharge calculated at upstream location (m^3/s)
T_h	time period at half discharge



Note that B is estimated initially from engineering judgement. This should be compared with the value obtained when calculating the estimate of floodwater depth. If the values differ significantly the calculation should be repeated using the newly calculated value of B. It is likely that one iteration will be sufficient to provide a reasonable value of B.

Following calculation of an attenuated value of Q_p, the associated new value of T_h may be found using the equation below. These new values of Q_p and T_h may then be used for subsequent attenuation calculations:

$$T_h(\text{new}) - T_h(\text{original}) \times Q_p(\text{original})/Q_p(\text{new})$$

The value of T_h therefore grows in proportion to the reduction in peak discharge.

Therefore, to calculate potential water levels at the intersection between valley zones:

For Zone 1 (immediately downstream of the dam):

1. Calculate Q_p at the dam and T_h from the flood hydrograph.

2. Estimate x – the distance from the dam to the end of the zone (that is, the zone length). Consider the path that the flood wave would take rather than following (measuring) a potentially meandering river route. During dam break the flow will not follow or be contained within the river channel.
3. Calculate the valley slope (for the zone) from map contours.
4. Estimate Manning's n for the valley considering the possible high depth of water.
5. Consider the potential flood water and make an initial estimate of the width of valley (B) that will be flooded, based on define base width and side slopes.
6. Calculate L_a using the equation above.
7. Calculate $Q_p(x)$ using the equation above.
8. Having calculated Q_p, the average depth of water at the zone intersection may be calculated using Manning's equation.
9. Compare the calculated and original values of B. If the difference is greater than 10% recalculate L_a, $Q_p(x)$ and the average depth of water using the calculated value of B. Repeat until the difference in values is less than 10%.
10. Plot a flooded outline for the zone using the water depths calculated.

Now repeat the above process across each zone using the values estimated from the previous zone as the starting point for the next.

Guidance on some of the detailed aspects of applying this methodology is given in Table 9.1.

Table 9.1 Guidance on detailed aspects of setting up Tier 2 rapid flood spreading

Manning's n roughness value	The selection of this value affects estimation of flow depth and rate of floe progression alone a valley. Although there is reasonable confidence in selecting appropriate values for river flow conditions (Chow 1986), it should be recognised that dam break flood conditions are very different. Under these conditions the extreme extent of flooding means that the flow is likely to pass through areas outside the normal flood plain, which may be heavily vegetated or developed. Under these conditions it is likely that the n value will be between 0.05 and 0.1, although the true value will be site-specific.
K factor to define rate of decay of floodwave	The attenuation length factor L_a represents the distance over which the initial breach discharge reduces to 37% of its initial value, this being directly proportional to the k factor. Although CIRIA C542 suggests a range of 1 to 10 (Hewlett et al. 2000), in some cases this has been found to give values of L_a which appear excessive, in that they exceed values typically obtained from standard analysis. It is therefore suggested that the factor k is adjusted to ensure that the downstream extent of total and partial destruction (q/w< $7m^3/s/m$ and $3\ m^3/s/m$ respectively) does not exceed the upper envelope curves given in Tarrent et al. (1994). These vary with height of dam, and for a 20m high dam, correspond to 15 and 10km respectively.
Attenuation factor	The attenuation length factor L_a represents the distance over which the initial breach discharge reduces to 37% of its initial value. Application of the theoretical derivation given in RMUKR often results in values of L_a which appear excessive, in that they exceed values typically obtained from standard analysis. As an interim measure it is suggested that the factor k is adjusted to ensure that the $_e$ limit of total destruction (DV 7m2/s; see Table 9.2.1) does not exceed 5kmand partial structural damage (DV 3m2/s) does not exceed 10km – based on summaries of detailed dam break shown on Figures 6 and

	7 of Tarrant et al. (1994) and applicable to dams less than 20m high. This approach is suggested as an interim solution until further research is carried out to provide an improved methodology. The limit of damage would be greater for larger dams. The suggested limit of extent of structural damage is that obtained from detailed dam break analysis.
Valley cross section	Some care and thought is required in setting up the trapezoidal cross-section used in the analysis, as this should be an average representing the length of river within each zone. Issues which need consideration are as follows. • The width of inundation should be consistent with the depth of inundation; it has been found helpful to measure the width between contours at say 5 and 10m above the valley floor, as a test of the geometry specified. • The length of the river bed should be consistent with the magnitude of flows; where high a straight line down the valley is reasonable; where the flows are more moderate the length should follow more closely the meandering path of the channel. This is important in terms of effective longitudinal slope. • Where dam break flows approach the magnitude of the fluvial 1,000 year flood, the published 1,000 year flood outline on the Environment Agency website can provide a useful check to the output from the rapid analysis.
Transportation embankments	These can have a major impact on the downstream extent of inundation, varying from total storage of the flood wave, to storage with consequential overtopping and secondary breach. (for example, see Brown et al. 2008). At Tier 2 the dimensions of transportation embankments are intended to be approximate (typically ±25% accuracy) to assist in the judgement as to whether the embankment would breach during the dam break flood. They would therefore be obtained from air and satellite photos available on the internet backed up by walk over survey in the field, rather than by needing to contact the owners of the infrastructure. It is good practice to include with the records of the risk assessment photographs of embankments (and associated cross drainage structures) which are likely to have a significant effect on flow conditions down the valley.

9.2 Step 2d – Consequences of failure

Flooding resulting from a dam break can impact on the downstream area in a variety of ways (such as injuries and fatalities, damage, disruption and loss of income) depending on the receptors located in the area of potential inundation. To account for these potential impacts, the possible effects of inundation on the receptors present (identified in Step 1b) need to be evaluated.

The following method includes guidance on evaluating the effects of flooding on:

- people
- economic activity
- the environment
- cultural heritage

Box 9.1 Assessing a range of impact on non-monetised receptors

The potential consequences of failure range from shallow flooding (say <0.5m deep) to very deep fast moving water carrying debris which will cause total structural destruction of two- and three-storey buildings. Although a simplified system can take this into account in assessing consequences to people and property, this is more difficult for other receptors which are not monetised, such as the environment and cultural heritage.

This guide (for Tier 2 analysis) has subdivided the potential for damage (impact) of several of the receptors into two categories, Extreme and Moderate, with the boundary based on the definition in Defra/Environment Agency (2006), with the boundary between the two hazard classes listed as being where Hazard = Depth $\times$ (Velocity +0.5) is greater than $3.5m^2/s$, and taken to be equivalent to a 5m high dam for the reasons given in section 20.1.4. The user may modify these to suit specific receptors. The overall consequence designation is the greater of the values for the two levels of impact.

9.2.1 Risk to people within the inundation area

People are at risk of suffering death or serious injury when flooding occurs.[7] People are unable to stand in deep or fast flowing floodwater. Once unable to stand they are at high risk of death or serious injury.

Adults are unable to stand in still floodwater with a depth of about 1.5m or greater (although this depends on the height of the person). The depth of flowing water in which people are unable to stand is much less. Some people will be at risk when water depth is only 0.5m if the velocity is 1m/s (about 2mph). If this is increased to 2m/s (4mph), some will only be able to stand in 0.3m of water. Most people will be unable to stand when the velocity is 2m/s and the depth is 0.6m.

Use the average water velocity and depth in each reach (derived in the preceding section) appropriate to the property group identified (which may include subdivision for position across the inundated area) to calculate average societal life loss, individual risk and property damage.

[7] Methods for analysing the secondary or indirect effects resulting from flooding that potentially pose a risk to life (for example, gas main explosion, accident on traffic diversion) are not considered in this guidance but should be considered on merit.

Table 9.2 Key parameters for assessing impact to people and property

Parameter	Suggested value for Tier 2	Justification
Minimum depth and velocity to create hazard	Neglect	This is a second order effect relative to the accuracy of dam break maps. Simpler to consider full extent of dam break maps
Level of damage to buildings	Inundation only: V <2m/s or DV < $3m^2/s$ Partial structural: V> 2m/s and $3m^2/s$< DV < $7m^2/s$ Destroyed: V > 2m/s and DV > $7m^2/s$	Binnie & Partners (1991)
Relationship of DV to Q/w	Q/w = 0.67 DV	Acknowledgment that d is maximum depth, whereas Q/w is a measure of average depth across the inundated area
Number of people present in each house	2.35	
Average occupancy in a building	Houses 80% Factory 55% Shops 50% Office 30% School 15%	Table 20.8
Non-residential building: • Number of people • Occupancy factor	• Determined from floor area, assuming $40m^2$ per person • 25%	Table 20.7
Population present on transport links	Table 9.3	
Fatality rate	Figure 9.1	Interim Guide (Brown and Gosden 2004), which is based on Reclamation DSO-99-06 (Graham 1999). Commentary in section 20.2.1.
Damage to houses/ non-residential property: • Inundation • Destroyed	£44,000 (£880/ m^2) £232,000 (£1,740/m^2)	Table 20.8 Table 20.9
Uplift on damage costs for cost of emergency services	5.6%	

Population at risk (PAR)

Although shallow water poses a low risk to individuals, it is often important to identify the total number of people in a threatened area that might need evacuation.

The purpose of this part of the assessment is to provide an estimate of the number of people likely to be present in the inundation area when the dam break flood arrives, for the consequences scenario(s) selected. Persons may be located in different locations which affect their vulnerability to floodwaters (inside or outside buildings, for example, or in a vehicle on a road).

PAR can be estimated using the assumptions given in Table 9.2.

Count or estimate the number of residential properties shown on a 1:25,000 scale map that are within the inundation area (counting individual properties if there are only a few but estimate broad numbers if there are a large number).

Where there are more than a few isolated properties, then estimate the number of residential buildings using one of the following:

- Divide the overall length of frontage of residential buildings on a street by the average plot width.
- Divide the overall area of residential development in hectares by the average gross plot size.

This may be supplemented by a site visit to verify properties at risk (normally carried out at the same time as a visit to confirm the features governing the potential extent of inundation).

Apply the same method as for residential properties to count the number of commercial properties in the inundation area (retail, factories and warehouses).

Table 9.3 provides a methodology and preliminary values for the population likely to be on transportation routes affected by a dam flood wave; averaged over a 24-hour basis. Clearly the actual number affected will vary with the time of day, being significantly greater during rush hour and summer evenings.

As well as those in the flood path at the time the flood wave arrives, some allowance could be made for additional vehicles, arriving after the dam break but before the road/ railway is closed, which may not stop without being affected by the flood wave. In the absence of published research on this it is suggested that the value given in Table 9.3 are doubled, where there is poor visibility and/ or high speeds that mean approaching vehicles are unlikely to detect the floodwater.

Table 9.3 Preliminary values for estimation of PAR on transportation links

	A road	Country lane	Footpath	Railway (main line)
Number of vehicles per day	12,000	100	24	150
Number of people/ vehicle	2	2	1	200
Average speed (kph)	80	50	3	140
Time to cross inundation zone[1] (minutes)	0.4	0.6	10	0.2
PAR in inundation zone when dam flood wave hits (averaged over 24 hours)	6.3	0.1	0.2	4.5

Notes: [1] For 500m wide inundation zone

Highest individual vulnerability (HIV)

One of the measures of risk from dam failure is the increase in annual chance of death of individuals who live or work in the inundation zone, due to the dam. This is the product of the annual chance (probability) of the dam failing and the individual vulnerability.

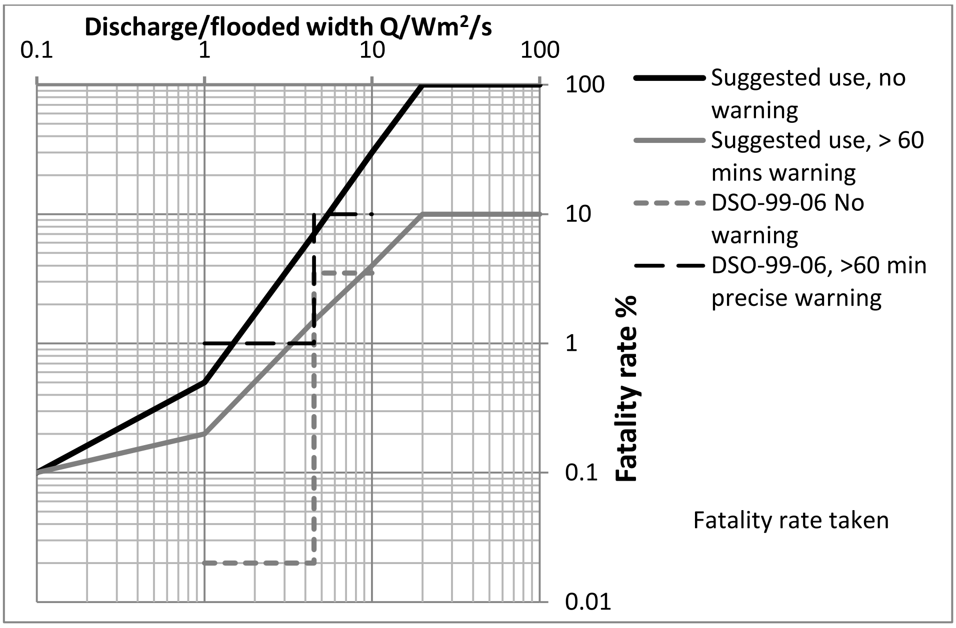
To calculate individual vulnerability in each reach (or population group):

i. Assess the hazard to an individual life from the hydraulic parameters of velocity and depth (see section 20.2.1) (that is, chance of death given dam failure) expressed as fatality rate. Read off the fatality rates from the graph in Figure 9.1 noting that the measure of forcefulness is the total discharge divided by the flooded width.

ii. Overlay this onto the exposure (percentage of time present) of hypothetical individuals (representative of the group) to determine the overall level of individual vulnerability, on an annual basis (suggested typical values are given in Table 9.2).

iii. Examine the results for all the reaches and take the highest value of fatality rate × occupancy to obtain the highest individual vulnerability.

Figure 9.1 Suggested relationship of fatality rate to force of water



Average societal life loss (ASLL)

The measure, average societal life loss, is the sum of product of the time averaged population and fatality rate of each population group, and thus a measure of potential overall impact on society. It should be noted that a value of 1.0 does not mean a specific individual but that, on average, a number of people each have some chance of death due to dam failure (this could be two people each with a 50% chance of death).

To determine average societal life loss:

i. Sum the product of the fatality rate (from Figure 9.1) and the assessed population at risk (from 9.2.1).

ii. Repeat for each population group and add them together to determine the average societal life loss.

Community health assets

There are a variety of community service and health assets (for example, hospitals, residential care homes, fire stations, prisons and waste management facilities) that could be adversely affected by a flood event. Many of these are identifiable from 1:25,000 scale maps and/or from a site visit to the area. Record the consequence designation (0–4) from Table 9.4 according to the type of community health assets and services in the inundation area.

Table 9.4 Consequence designation for community health assets

Community health assets		
Level of impact		**Consequence magnitude**
Moderate (that is, q <2.5m^3/s/m)	**Extreme(that is, q >2.5m^3/s/m)**	**Level**
None	None	0
Any CH3 asset type	None	1
Residential homes and any CH2 asset type in urban area Any power supply	Any CH3 asset type	2
Any one CH1 asset type in remote rural areas	Residential homes and any CH2 asset type in urban area Any power supply	3
More than one asset of type CH1	Any 1 CH1 asset type or more	4

Key:
CH1= Hospital, ambulance depot, residential home, health centre/clinic, police, fire station
CH2 = Education facility, prison, power supply (for example, transformers)
CH3 = Pharmacies, post offices, water pumping and waste management sites

9.2.2 Economic activity

Economic damages arising from flood inundation can arise from a number of sources including direct external damage to buildings, costs of internal remediation and loss of belongings from residential property as well as loss of goods in, and services from, commercial premises. Although difficult to value, loss of income from productive land should also be considered. Transport disruption can have widespread economic impacts that are virtually impossible to value, although direct damage to the infrastructure and costs to railway and airport operators of disruption of services can be more easily estimated.

Residential and commercial property

To assess the total amount of damages, use the depth and velocity for each inundation zone (see section 9.1) to calculate total damages for the properties within them.

For conventional UK property it may be assumed that, when the product of velocity and depth (VD) is less than 2m^2/s, damage is limited to inundation damage, while when it exceeds 7m^2/s, the building is destroyed, with partial structural damage in-between (taken as 50% of write-off damage).

Methods of estimating property damage values are given in section 20.2.2, but at 2011 values average inundation and structural damage to residential properties may be taken as those shown in Table 9.5.

Table 9.5 Average annual property damage values (2011)

Flood impact	**Residential property**	**Non-residential property**
Inundation damage	£44,000 per house (at 3m depth flooding)	£881/m^2 (at 3m depth flooding)
Collapse/destruction (>7m^2/s)	£233,000 per house	£1,740/m^2

Where required more detailed costing of damages is possible. Further guidance can be found in other guides such as FHRC (2010).

Transport

Transport networks and associated assets can be damaged and/or disrupted by flooding. The impacts will depend on various factors such as the type of asset (road, railway, airport), the time of inundation, the traffic frequency/level of use, and the options for alternative routing. Consider such receptors in the inundation area that could be affected.

Record the consequence designation (0-4) from Table 9.6 according to the type of transportation assets in the inundation area.

Table 9.6 Consequence designation for transportation assets

Transportation assets	**Consequence magnitude**
(Any flood intensity/velocity)	**Level**
None	0
Any B and minor roads unless in very remote areas	1
B and minor roads in remote areas All A roads unless in remote areas	2
Airports, railways, motorways. A roads in remote areas	3
>1 of any of airports, railways, motorways. A roads in remote areas	4

If required, costs of impacts on transport can be calculated. If required see FHRC (2010) for further guidance.

Agriculture

Agricultural land, crops, livestock equipment and buildings can all be affected by floodwater. The extent of impacts and damages will depend on depth, velocity, duration and speed of onset of sudden inundation. If thought to be significant or required, costs of impacts on agricultural receptors can be calculated. If required, see section 20.2.2 for guidance on impacts on agricultural land and Chapter 9 of FHRC (2010) for detailed analysis of damages.

As a minimum for a Tier 2 analysis, record the consequence designation (0–4) from Table 9.7 according to the main Agricultural Land Class (ALC) in the inundation area. Definition of ALCs can be found in section 20.2.2; further information on Agricultural Land Classification can be accessed via the Natural England GIS digital boundary datasets entry web page (www.gis.naturalengland.org.uk/pubs/gis/gis_register.asp).

Table 9.7 Consequence designation for agricultural land types

Agriculture (Agricultural land use types)		**Consequence magnitude**
Level of impact		
Moderate (that is, <25% of area affected)	**Major (that is, >25% of area affected)**	**Level**
None	None	0
ALCs 4 and 5	ALC 5	1
ALCs 3a and b	ALC 4	2
ALC 2	ALCs 3a and b	3
ALC 1	ALCs 1 and 2	4

9.2.3 The environment in the inundation area.

Designated areas

Habitats and species can be adversely affected by flooding. Record the consequence designation (0–4) from Table 9.8 according to the type of conservation/protected area and habitats and species in the inundation area (that is, Local Nature Reserve, Site of Special Scientific Interest, Special Protection Area, Ramsar, Natura 2000 and so on).

Lists of designated areas can be sourced via the Natural England website (www.naturalengland.org.uk/ourwork/conservation/designatedareas/default.aspx).

A list of principal habitats and species of importance in England can also be found on the Natural England website (www.naturalengland.org.uk/ourwork/conservation/biodiversity/protectandmanage/habsandspeciesimportance.aspx).

The equivalent sources of information for designated areas in Scotland and Wales are the Scottish Natural Heritage website (http://gateway.snh.gov.uk/sitelink/index.jsp) and the Countryside Commission for Wales website (www.ccw.gov.uk/default.aspx?lang=en).

Table 9.8 Consequence designation for areas designated as conservation or protection areas for habitats and species

Environmental impact		
Level of impact		**Consequence magnitude**
Moderate (that is, <25% of area affected)	**Major (that is, >25% of area affected)**	**Level**
None	None	0
Local Nature Reserves	None	1
Statutory designations and designated sites/protection areas **not** containing protected habitats or species	Local Nature Reserves	2
Statutory designations and designated sites/protection areas **containing** protected habitats or species	Statutory designations and designated sites/protection areas **not** containing protected habitats or species	3
Internationally designated sites (Ramsar, Natura 2000, SSSI)	Statutory designations and designated sites/protection areas **containing** protected habitats or species Internationally designated sites (Ramsar, Natura 2000, SSSI)	4

Report the designated areas as area affected in hectares by measuring off the 1:25,000 scale Ordnance Survey map, or take from site citation on The MAGIC web-based interactive map service (http://magic.defra.gov.uk).

9.2.4 Cultural heritage

Cultural heritage including assets such as historic buildings, parks and gardens, ancient monuments and so on can be damaged by floodwater. Such sites and monuments are often shown on 1:25,000 scale maps. The National Heritage List for

England can be found on the English Heritage website (http://list.english-heritage.org.uk/). The equivalent sources of information in Scotland and Wales are the Historic Scotland website (www.historic-scotland.gov.uk) and for the Historic Wales website (http://jura.rcahms.gov.uk/NMW/start.jsp).

Record the consequence designation (0-4) from Table 9.9 according to the type of cultural heritage asset in the inundation area.

Table 9.9 Consequence designation for cultural heritage

Cultural heritage		
Level of impact		**Consequence magnitude**
Moderate (that is, q <2.5m^3/s/m)	**Extreme (that is, q >2.5m^3/s/m)**	**Level**
None	None	0
Grade II listed buildings, registered parks and gardens	None	1
Grade II* listed buildings, registered parks and gardens	Grade II listed buildings, registered parks and gardens	2
Grade I listed buildings, registered parks and gardens Scheduled ancient monuments and archaeological sites	Grade II* listed buildings, registered parks and gardens	3
UNESCO World Heritage Sites	Grade I buildings/ parks and gardens UNESCO World Heritage Sites	4

9.2.5 Overall consequences of failure

Collate the above base measures and consequence magnitudes derived above into a table similar to that as shown in Table 9.10. For the basic analysis, with a single overall probability of failure this would be plotted with the worst of the two scenarios – normally the rainy day.

To limit the number of variables to be considered in the risk assessment, the measures of risk to life should carried forward and a check made to determine if any other impacts are likely to be more significant than risk to life in determining the need for risk reduction measures. This would only normally apply where there were no people resident downstream but there was likely to be damage to nationally important assets.

Table 9.10 Example of a table of collated consequence designations

Receptor category	Attribute	Consequence scenario	
		Sunny day	Rainy day
Base measures of consequences			
Human health	PAR – maximum (used for planning evacuation)	35	80
	PAR – time averaged	23	57
	Highest individual vulnerability (HIR)	0.2	0.5
	Average societal life loss (ASLL)	4.6	28.5
Other indicators of consequences			
Human health	**Community health assets**	0	2
Economic activity	Residential properties	£156,300	£986,200
	Non-residential/ commercial properties	£990,000	£1.5 million
	Transport	2	2
	Agriculture	2	3
Environment	Habitats and species	0	1
Cultural heritage	Designated sites, listed buildings, scheduled monuments	1	1
Consider if any other risks are likely to be more significant than life loss.		Tolerability of risk likely to be determined by potential loss of life	Carry risk to agriculture forward (3)

9.3 Steps 2e and 2f – Determine level of risk and review

9.3.1 Step 2e – Determine level of risk

Risk is a function of the failure scenario multiplied by the consequences of the failure. Thus the analysis process now estimates the risk arising from the different threats, failure modes and consequences.

Societal risk

Plot the ASLL risk (calculated in section 9.2.1) on an F-N chart (Figure 9.2) against the overall probability of failure from section 8.4. (Risk data should also be assessed in relation to other impacts as shown in Table 9.2, or as defined by the scoping in Step 1c.)

The societal risk point plotted on the F-N chart will fall into one of three categories as divided by the 'ALARP' boundaries in Figure 9.1. This means that the social risk will classed as being one of the following:

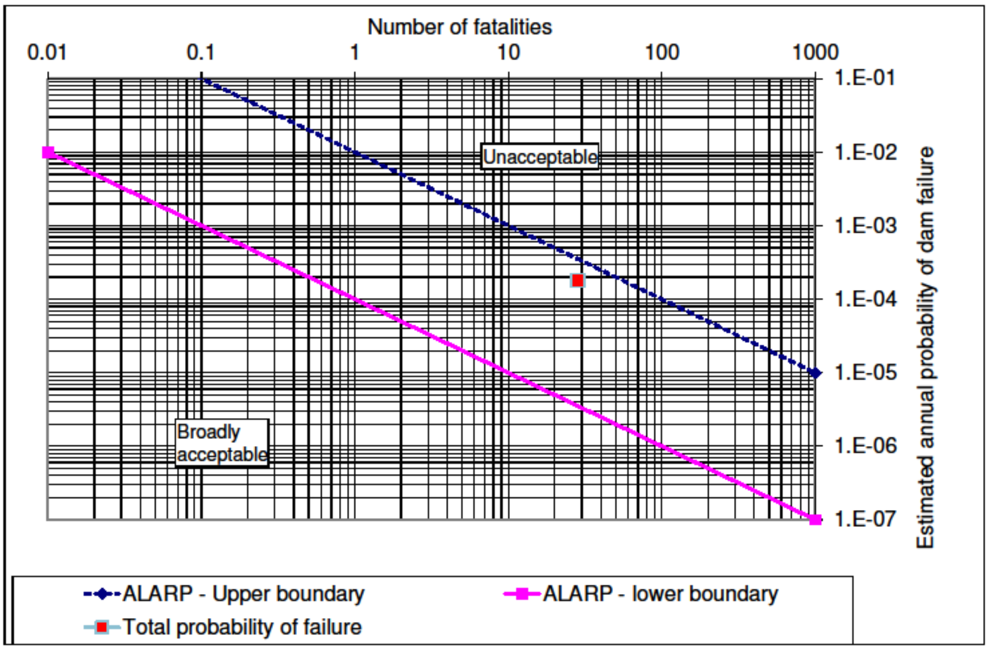
a. 'Broadly acceptable' – risks compared with those that people live with every day, and that they regard as insignificant and not worth worrying about (for example, health risks associated with using mobile phones)

b. 'Unacceptable' – risks are generally believed by individuals and society to be not worth taking regardless of the benefits (for example, building residential areas on toxic landfills)

c. ’Within the range of tolerability’ – individuals and society are willing to live with the risks so as to secure certain benefits, provided that they are confident that they are being properly managed, and that they are being kept under review and reduced still further if and as practicable.

These categories are adapted from HSE (2001) and Le Guen (2010).

Figure 9.2 Example of a simple F-N plot



Individual risk

Individual risk is the increase in chance of death per year due to the presence of the dam, to those living downstream, It is calculated as the product of the ‘individual vulnerability (percentage of time present times fatality rate if the dam failed) and the overall portability of failure. An example is given in Table 9.11.

Table 9.11 Example table summarising some of risk outputs (combinations of probability and consequences)

Feature	**Value**	**Comment, source**		
Total probability of failure	1.8 E-4	Table 8.15		
Consequences of failure		**Risk**		
Parameter	**Value**	**Units**	**Value**	**Units**
Average social life loss	28.5	Societal life loss per year	5.1E-03	Lives per year
Individual vulnerability	50%	Individual risk of death per year	9.0E-05	Chance per year
Economic damage to third parties	£1.5 million	Damage to third parties	£270	£ per year
Other: specify Agricultural land	3	Risk to agriculture (from matrix in Table 9.7)	Plot on matrix in Figure 9.3	Qualitative

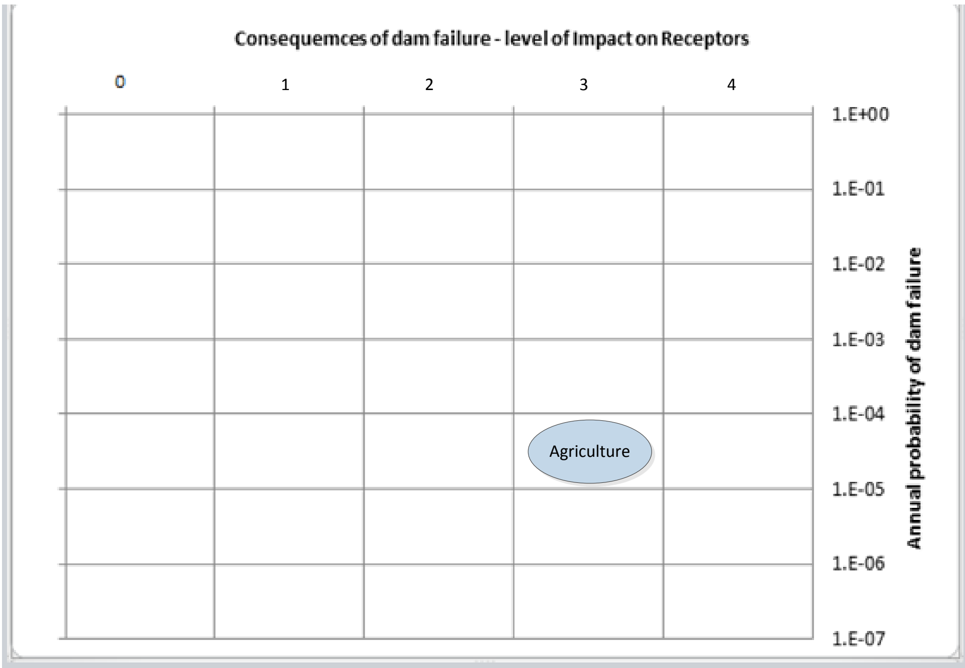
Economic damage to third parties

This is similarly determined as the product of the likely damages if the dam failed and the overall portability of failure. In simple terms it could be seen as an annual risk premium.

Other receptors

The other consequences of dam failure are not easily quantifiable, so it is suggested that they are summarised and assessed by plotting on a matrix similar to Figure 9.3. This can be useful for portfolio risk assessment where the consequence scores from several reservoirs can be plotted on one graph for comparison and quick reference.

Figure 9.3 Displaying other consequences of failure



9.3.2 Step 2f – Review outputs

It is important to review the outputs of the consequence analysis.

- Are all important receptors accounted for in the assessment?
- Do the results look credible/realistic? (see Section 15.2.4, and additionaly for concrete dams Section 17.5.5)
- Does the analysis need revisiting or refining with better information?
- Where could it be improved?

Conduct a critical review of the outputs, considering whether it can be carried forward, or whether any of the aspects in Step 2 should be refined.

This could, for example, include the need for more accurate data, or moving to a higher tier (and hence complexity) of analysis. Issues to consider include:

- What governs the overall consequence scenario?
- What governs the total probability?

- Do the risk levels look about right?
- Where are the gaps and what do I need to know more about?

9.3.3 Optional – Estimate range of uncertainty

It is recommended that a formal assessment and record of confidence in the assessment of risk is carried out. You can indicate your level of confidence in the assessment of risk (the likelihood of the event and failure scenarios and the magnitude of potential consequences you undertook in Step 2e) using the categories: ‘Very confident’, ‘Confident’, or ‘Not confident’. In addition a sensitivity analysis could be undertaken and documented to supplement this analysis.

Consider each combination of failure and consequence scenarios (that is each point you have plotted on the F-N chart and allocate one of the following levels of confidence that reflects your judgment.

- **Very confident:** for example, you are very confident that you have captured the likelihood of the hazard(s) and the magnitude of the consequences accurately in the risk assessment
- **Confident:** for example, you have captured the hazards and consequences in the risk assessment but are uncertain about the likelihood of the hazards and/or the magnitude of the consequences
- **Not confident:** for example, you are not certain that you have captured the hazards and/or the consequences sufficiently well in the risk assessment.

10 Tier 2 – Step 3 Risk evaluation

Step 3	Risk Evaluation	Step 3a	Review tolerability of risk assessment
		Step 3b	Review options to reduce risk
		Step 3c	Proportionality
		Step 3d	Other considerations
		Step 3e	Review and make recommendations

Risk evaluation is the process of examining and judging the significance of estimated risk (that is, is the risk tolerable or not) and also the consideration of what the costs and benefits are of various options to reduce the risk.

10.1 Step 3a – Review tolerability of risk

i. Consider the estimated total probability of failure (from Step 1c). Is it higher or lower than the individual risk limit value of 1 in 10,000 per year?

If the estimated total likelihood of failure is less than the individual risk (IR) limit value, assign a 'Yes' outcome (or and an apparent 'A Yes' outcome). If the range exceeds the IR limit value, then assign a 'No' (or an 'A No') outcome.

NB: The risk assessment may need to be improved to satisfy confidence and defensibility requirements of the owner/undertaker or the inspecting engineer if a 'Yes' outcome is assigned.

ii. Consider the societal risk (SR) Limit as shown by the sloping line on the F-N chart in Figure 9.2.

If the risk plotted on the F-N chart (which represents the estimated societal risk) is below the upper ALARP boundary then assign a 'Yes' outcome (or an apparent outcome as 'A Yes' if confidence in the risk assessment needs to be improved to satisfy requirements of the owner/undertaker, the inspecting engineer and other concerned stakeholders).

If the risk plotted on the F-N chart is above the upper ALARP boundary then assign a 'No' outcome (or an apparent outcome as 'A No' if confidence in the risk assessment satisfies the requirements of the owner/undertaker, the inspecting engineer and other concerned stakeholders.

iii. Is the annual risk cost, in terms of annualised damages per year acceptable to the owner?

Refer to Table 9.11. This table presents the risk based on total annual probability, and likely consequences.

iv. Are the other potential (non-monetary) consequences of failure tolerable?

Refer to Table 9.2 (the risk having been read off Figure 9.2).

v. For those measures of risk which are intolerable or ALARP, consider the causes (drill down into risk assessment).

Review both the build-up of likelihood of failure (Chapters 7 and 8 as applicable) and build-up of consequences (Chapter 9) to understand what is governing both parameters, and thus what is causing the intolerability.

Table 10.1 Example table summarising tolerability of risk to life

Risk			Tolerability
Units	**Value**	**Units**	
Societal life loss/ annum	5.1E-03	Lives per year	ALARP
Individual risk of death/ year	9.0E-05	Chance per year	ALARP

10.2 Step 3b – Review options to reduce risk

Consider practical options exist to reduce the risk and the costs and benefits of these in terms of the reduction in risk.

i. Identify what practical options exist to reduce risk. The types of options which are normally available are summarised in Table 6.2.

ii. Estimate the potential change in risk, and associated present value cost, **or** repeat the risk assessment for a scenario where the candidate works have been completed, to estimate the benefits in terms of the reduction in risk.

Present the options and their estimated effect on risk reduction in a table. An example is shown in Box 6.2.

Estimates of cost, at this feasibility stage, should include an allowance for optimism bias, which is typically taken as 30% of project cost at feasibility stage (HM Treasury 2003, updated 2011). Also, present value is approximately 30 times annual cost (when using government discount rates).

10.3 Step 3c – Proportionality

Consider whether the costs of the measures to reduce the risk are proportional to the potential reduction in risk achieved by those measures. This is normally achieved by calculating the cost to prevent a fatality (CPF) and comparing this with the value of preventing a fatality (VPF). At its simplest, where the CPF is less than the 'value of preventing a fatality' (VPF), then the candidate works would be proportionate risk reduction measures, while where CPF exceeds VPF, then the cost is disproportionate.

Calculating the cost to prevent a fatality is summarised as follows:

$$\text{CPF} = \frac{\text{Equivalent annual cost of risk reduction measures} - \text{Present Value } (\Delta \text{Pf} \times \text{Damage})}{\text{Present value } (\Delta \text{Pf} \times \text{likely loss of life (LLOL)})}$$

where ΔPf is the change in annual probability of failure due to the proposed risk reduction works.

NB: Only use the probability associated with the remediation (that is, the probability of failure which it affects) and not the total probability for the dam.

Costs should be estimated realistically. It is recommended (Defra 2003) that, at the pre-feasibility stage, an optimism bias of 60% is added to the best estimate of total cost. This is based on experience of total project out-turn costs against the pre-feasibility estimate.

A discount rate (currently recommended[8] as 3.5%) should be used to calculate the Present Value result in the present value being about 30 times the equivalent annual cost. This is assuming risks are discounted over a 100-year period, using interest rates set by the Treasury, which amounts to a factor of 30 on annual risk cost. The criteria for economic evaluation should be agreed with the project sponsor, as some reservoir owners may be required to use different discount rates, for example, if subject to Ofwat funding.

The value that should be assigned to VPF is a difficult decision and includes consideration of:

- direct costs (measurable) such as the earning potential of the victims, injury and long-term health impairment of other victims not included in the LLOL value, and emergency services costs
- indirect (business losses)
- intangibles (psychological impact on people, environmental damage) – it could be argued that a value should be assigned to the intrinsic value of a human life (irrespective of age, health, education and so on)

The Department for Transport's assessed VPF for road and rail for 2010 was £1.7 million (Department for Transport, 2012).

However, 'gross' disproportion is required before ALARP is satisfied and defines a 'proportion factor' defined as:

$$\text{Proportion factor (PF)=} \quad \frac{\text{Cost to prevent a fatality (CPF)}}{\text{Value to prevent a fatality (VPF)}}$$

The purpose of a proportion factor 'grossly' greater than unity is to allow for the imprecision of estimates of costs and benefits and also to ensure that the duty holder robustly satisfies the ALARP principle.

HSE guidance on what constitutes a reasonable proportion factor (ALARP Suite of Guidance, www.hse.gov.uk/risk/theory/alarp.htm) includes the statement that:

> 'NSD [Nuclear Safety Directorate] takes as its starting point the HSE submission to the1987 Sizewell B Inquiry that a factor of up to 3 (that is, costs three times larger than benefits) would apply for risks to workers; for low risks to members of the public a factor of 2, for high risks a factor of 10.[9]

Hughes and Gardiner (2004) present a disproportionality factor which varies with probability of failure (POF), from 3 at POF of 10^{-6} to 10 at POF of 10^{-4}.

[8] See *The Green Book – Appraisal and Evaluation in Central Government*, HM Treasury, 2003, updated 2011.

[9] HSE principles for Cost Benefit Analysis (CBA) in support of ALARP decisions (www.hse.gov.uk/risk/theory/alarpcba.htm)

Box 10.1 Example output assessing proportionality

Proportionality		
Example works through one method to reduce the risk.		
ASLL for no warning Economic damages (£) VPF (£M)	7.9 £3,576,000 £1.70	
Option to reduce risk:	Inspection of gunite (10 yearly)	
Probability of failure		
Before mitigation With mitigation	1.00E-04 1.00E-05	
Present value of overall project cost (£k) (=30 x annual cost)	£1,000	
Annual cost of damages		
AP existing * Economic damage AP after works * Economic damage	£358 £36	
Annual cost of damages of existing - Annual cost of damages after works	£323	
ASLL per annum		
AP existing * ASLL per annum AP after works * ASLL per annum	7.90E-04 7.90E-05	
AP existing - AP after works	7.10E-04	
Cost of preventing a fatality		
(Present Value/30 - Presernt Value of reduction / 1000) / Present value of saving lives *1000	£4.2M	including damages
(Present Value/30) / Present value probability reduction * 1000	£4.7M	Life only
Proportion factor		
Cost of preventing failure / VPF	2.5	including damages
Cost of preventing failure / VPF	2.8	Life only

10.4 Steps 3d and 3e – Other considerations, review and recommendations

10.4.1 Step 3d – Other considerations

As well comparing the benefits and costs of potential risk reduction measures, the risk assessment should consider the following questions for any practical options that can be identified to further reduce the risk.

1) **Have either the IR or SR evaluations in Step 3b resulted in a 'No' or 'A No' outcome?**

Compare the level of risk with the limit guidelines – if either the individual risk or societal risk evaluations in Step 3b resulted in a 'No' or an 'A No' outcome, then ALARP has not be demonstrated unless there are extraordinary circumstances preventing these limit guidelines being met by any practical risk reduction measures.

2) **Does the risk assessment satisfy the confidence and defensibility requirements of the owner or undertaker and any other stakeholders?**

As appropriate, ensure that any societal concerns are adequately addressed. Stakeholders, including those who would be affected by dam failure or dam repairs should be consulted and their concerns addressed. The outcomes of this evaluation can be indicated by 'Yes' or 'No', or 'A Yes' or 'A No' if the risk assessment does not satisfy the confidence and defensibility requirements of the owner or undertaker, and any other stakeholders.

3) **Have all the risk guidelines identified in the pre-assessment (see Chapter 2) been adequately addressed?**

Additional risk criteria (as identified in the pre-assessment) can be listed and if appropriate the outcomes of evaluating them can be indicated using 'Yes', 'A Yes', 'A No', or 'No'.

4) **Does the dam meet published engineering standards for the UK?**

Confirm whether the dam has been assessed against published standards such as floods and seismic design, and if so did it meet the published standards, or are there outstanding deficiencies? (Refer to section 2.4.6.)

5) **Have any deficiencies identified in previous studies been addressed?**

Confirm whether the risk assessment has taken into consideration previously identified deficiencies (that are unresolved) and whether or not these have been addressed in the risk assessment.

10.4.2 Step 3e – Review and make recommendations

Where possible compare the outputs of the risk assessment with similar dams.

Bring together both the risk analysis (including the quantitative evaluation of risk reduction measures (Step 3c) and considerations of other factors (Step 3d) to make a decision recommendation with an accompanying justification (refer to the list of decision issues identified in the scoping step).

11 Tier 3 – Step 1 Risk identification

Step 1	Risk Identification	Step 1a	Failure modes identification (FMI)
		Step 1b	Identify potential consequences
		Step 1c	Review; scope risk analysis

There are many ways in which failures of dams can occur. Failures occur through the realisation of a combination of a threat and a failure mode. Mechanisms by which failure can occur can happen rapidly or slowly overtime. The probability of failure associated with specific combinations of threat and failure mode varies; some modes of failure are more common than others.

11.1 Step 1a – Identification of potential failure modes

For Tier 3 it is suggested that failure modes identification should be performed by a small team. At a minimum this team should consist of two inspecting engineers, the supervising engineer and the reservoir keeper.

All team members should review all available information on the reservoir system before commencing the failure modes identification process. This information should include evidence for any potential failure modes based on intrinsic condition of the reservoir system or indicators of failure modes. For Tier 3 consideration should be given to:

- collecting key additional data such as up-to-date topography, exploratory borings and soil testing
- performing any supplemental analyses such as seepage and stability study, updated design flood estimate

Conduct a team site inspection and document intrinsic condition of the reservoir system with an eye toward identifying evidence for any internal threats and potential failure modes. Refer to the lists in the Sections 4.4 and 8.5.

11.1.1 Structured identification of potential failure modes

List all significant external threats (initiating events or sources) that could initiate a failure mode. Refer to the list in Table 7.2 but do not be limited to this list. (Further references that can assist in the identification of failure modes are given in Table 7.2.)

List all components of the reservoir system and their functional roles in preventing dam failure and interdependencies with other components.

Based on the functional understanding of all components of the dam system, identify all potential failure modes over the entire range of possible magnitudes of each external threat with consideration given to the evidence based on indicators, intrinsic conditions, and outcomes of any condition assessment undertaken (see section 2.4.7).

Descriptions of failure modes should differentiate

- threat (initiation)
- failure mode (progression)

- breach

Thus, for example, failure by sliding can occur due several threats, namely elevated reservoir level in flood, earthquake or foundation deterioration. Description of the failure mode therefore needs to include all these elements of the overall failure process

11.1.2 Identification of credible and significant failure modes

Perform and document a preliminary classification of potential failure modes as credible or not credible and all credible failure modes as significant or not significant failure modes.

Prepare a detailed description of each credible and significant failure mode from initiation (threat) to breach, including more and less likely factors and areas of significant uncertainty.

Prepare a final list of credible and significant failure modes, including documentation of any changes in classifications.

Develop preliminary event trees for all credible and significant failure modes for use in the risk analysis.

Complete documentation of failure modes identification, including a list of major findings and understandings gained.

11.2 Step 1b – Identification of potential consequences

This sub-section considers the potential failure modes and uses this to scope the assessment of potential consequences of dam failure for a Tier 3 analysis.

Entry level would normally be carried out using GIS software to make use of property databases, with fatality rates assessed using velocity and depth from legacy dam break maps. More advanced consequence assessment could include one or more of:

- two-dimensional (2D) dam break, adjusting fatality rate for water depth at the property
- simulation of human response to evacuation/ rising flood waters using software such as LIFESim/LSM as appropriate

11.2.1 Consequence scenarios

The key issues defining the consequence scenario(s) to be analysed were summarised in section 7.2 for a Tier 2 analysis; they would normally be similar for an entry level Tier 3 analysis. Further commentary on possible alternative assumptions is given in section 20.1.

11.2.2 Incremental damage

For Tier 3 analysis it is normal to assess the incremental consequences of dam failure, which therefore requires assessment of the effects for the scenario of no dam failure. For proposed dams it is up to the user whether this is with dam absent (pre dam construction), or whether the dam is present but assumed not to fail. For existing dams it would normally be with the dam in place, such that any benefits in terms of attenuation of fluvial floods with no dam failure forms the baseline.

For failure due to flood it is suggested that a range of floods are considered, as the largest incremental effect may not be at PMF but at some smaller flood.

11.2.3 Geographical extent

As the analysis is carried out using GIS there is no need for the subdivision of the inundation zone into a limited number of reaches, with sub-division of features impacted for reporting purposes being a user choice, the risk assessment requiring cumulative impacts.

11.3 Step 1c – Review of outputs and scope risk analysis

11.3.1 Review of outputs

Conduct a critical review of the outputs from Steps 1a and b, considering whether they can be carried forward, or whether any aspects of the assessments should be refined.

If there is insufficient information available about the condition of the dam to inform the risk assessment then consider performing a condition assessment.

Condition assessments should consider published guidance to panel engineers and accepted good practice. Consider condition of the dam for normal operation, for flood conditions and for earthquakes. Recommended elements for which to consider condition are listed in Table 16.2.

A key issue is the availability of data on the dam and their quality. If there are not enough data available to complete Step 1a then a critical examination should be carried out of the data in relation to the failure modes for which the probability of failure is to be estimated, and a record made of what exists and its quality. This assessment should normally be carried out by an individual, with a second individual carrying out quality checks.

A Tier 3 consequence assessment would normally require GIS data on property locations, numbers and types, as well as data on the other features which would be impacted by dam failure. Table 20.12 lists available GIS datasets of potential receptors.

12 Tier 3 – Steps 2a-c Likelihood of failure

Step 2	Risk Analysis	Step 2a	Likelihood of failure due to internal threats
		Step 2b	Likelihood of failure due to external threats
		Step 2c	Dambreak and flood routing

12.1 Likelihood of failure

Threats which could lead to dam failure can be subdivided into external threats and internal threats. The latter are those where the root cause of failure is within the body of the dam, or its foundation, caused for example by deterioration or ageing. These types of failure comprise around half of the causes of failure of dams in service (Brown and Tedd 2003). This may lead directly to failure under constant load, or may weaken the dam to such an extent that it fails rapidly when subject to a change in external load. The likelihood of failure due to internal threats is estimated using data on historic frequency of failure, as being the most reliable method.

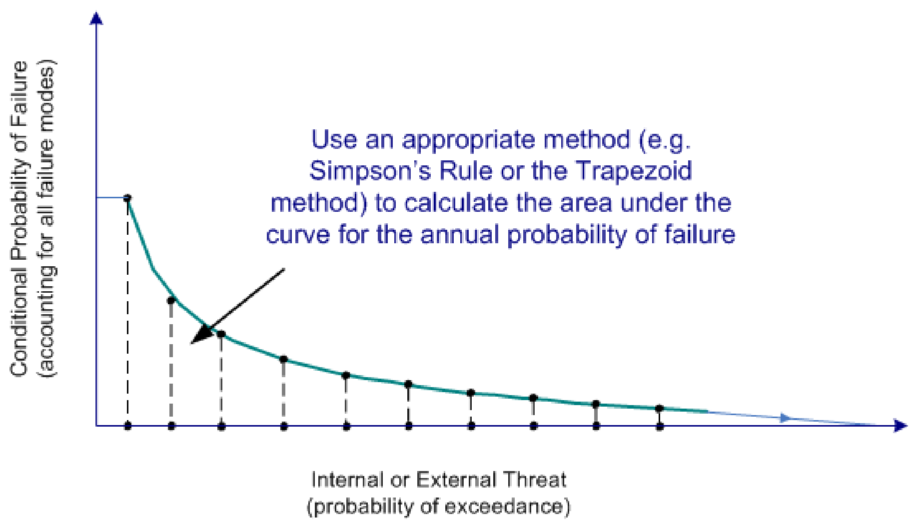
The likelihood of failure due to external threats is assessed using analytical methods, whereas the likelihood of failure is assessed using event trees. Further details are provided in Chapter 8.

Plotting the conditional probability of failure against the probability of the driving internal or external threat enables annual probability of failure (APF) to be calculated.

To calculate the annual probability of failure the chance of the threat must be combined with the conditional chance of failure given the threat.

i. Use appropriate means of calculating the area under the curve (see Figure 12.1) such as the Trapezoid method, Simpson's Rule or similar to yield the annual probability of failure.

Figure 12.1 Plotting the conditional probability of failure against the probability of the driving internal or external threat



12.1.1 Step 2a – Likelihood of failure due to internal threats for embankment dams

The physical mechanisms controlling initiation and the rate of development of internal threats are still not fully understood, and for internal erosion are controlled by all of the three elements of material susceptibility, stress state and hydraulic load. Thus it is necessary to consider both the **intrinsic condition,** which is how the dam was built, and the **current condition,** which reflects the current stress state and voids ratio in the dam and thus reflects ageing of the dam and how the dam has reacted to load. As the root cause is within the body of the dam it is difficult to measure what is happening inside the dam such that assessment has to rely on external features/measurement, any monitoring of parameters within the dam and knowledge of performance of similar dams.

The identification of potential failure modes is described in Step 2, while this section describes procedures to assess the probability of failure due to internal threats, as part of a Tier 3 risk assessment.

At present, although methods exist to calculate the probability of failure due to external threats (for example, slope instability), using Monte Carlo analysis of variable impounded water level and soil parameters, there is no equivalent analytical method to estimate the probability of failure due to internal threats. It is therefore suggested that estimation of the probability of failure of internal threats is carried out using event tree methods, as these promote thinking through and understanding the mode of failure, and whether it would lead to damage to the structure or could lead to failure (release of the reservoir).

Further information on alternative methods of estimating the probability of failure due to internal threats is given in the supporting information in section 17.3.

Tier 3 assessments are normally carried out by a team, often including an external facilitator.

At Tier 3 there are two established methods for estimating the probability of failure using event trees as shown in Table 12.1. Both rely on defining event trees to represent the internal failure process (see example in section 8.3.1) and then assigning probabilities to each branch of the event tree, with each method providing guidance on suitable base probabilities and factors to be considered in estimating branch probabilities from the experience and judgement of the authors, and the judgment of the assessor. These should be supported by an in-depth description and understanding of the physical processes involved, using the ICOLD Bulletin on internal erosion (ICOLD 2012).

Two event tree methods for the estimation of the probability of failure due to internal erosion (see Table 12.1) are described in the Seepage and Piping Toolbox (SPT) (Fell et al. 2008a)[10] and Chapter 24 of Reclamation's Best Practices Training Manual (BPTM) (Reclamation 2010a). The SPT utilises historical information on incidents and failures drawn from available information from many countries. It provides a systematic and detailed process for estimating branch probabilities for the generic event tree shown in Figure 12.2.

[10] Methodologies described in *A Unified Method for Estimating Probabilities of Failure of Embankment Dams by Internal Erosion and Piping. Draft Guidance Document dated August 21, 2008* (Fell et al. 2008a) are under evaluation. Reclamation and USACE have not endorsed it for estimating the probability of failure of embankment dams by internal erosion and piping. Neither agency will provide anyone holding a copy with updates or corrections.

Figure 12.2 Generic event tree from the Seepage and Piping Toolbox (Fell et al. 2008a)

Reservoir rises

Initiation – flaw exists (1) (2)

Initiation – erosion starts

Continuation – unfiltered or inadequately filtered exit exists

Progression – roof forms to support a pipe

Progression – upstream zone fails to fill crack

Progression – upstream zone fails to limit flows

Intervention fails

Dam breaches (consider all likely breach mechanisms)

Consequences occur

(1) A ‘flaw’ is a continuous crack or gap, poorly compacted or high permeability zone in which a concentrated leak may form.

(2) For backward erosion piping (BEP), no flaw is required, but a continuous zone of cohesionless soil in the embankment or foundation is required.

The SPT also contains tables that provide guidance on the estimation of conditional probabilities. These tables were developed to model the physical processes so far as practical. The probabilities were estimated using the expert judgment of workshop attendees. Where practical, the probabilities were anchored to historic data.

The BPTM approach uses a similar event tree except that the second and third nodes are combined into a single node, ‘Initiation – erosion starts.’ The BPTM provides suggested ‘starting points’ or base annual probabilities for the probability of the initiation of internal erosion in an inventory of dams similar to the SPT’s. It also contains guidance on the estimation of branch probabilities but in less detail than the SPT.

It is suggested that the user selects one of these methods and has the relevant guidance and software to hand before starting the assessment. A key point in the application of the methods is the extent to which the guidance on selecting conditional probability should be adjusted for UK dams. This is discussed further in section 17.5.2.

Table 12.1 Published event tree methods for estimation of probability of failure due to internal threats

Method	Reference	Available from	Number of potential failure modes
Seepage and Piping Toolbox (SPT)	Fell et al. (2008a)	US Corps of Engineers	28 (Table 3.5)
Chapter 24 of Dam Safety Risk Analysis. Best Practices Training Manual, (BPTM) (Note 1)	Reclamation (2010)	www.usbr.gov/ssle/damsafety/Risk/methodology.html	Five groups

Notes [1] This is a development of the preceding ‘Risk Analysis Methodology, Appendix E: Estimating Risk of Internal Erosion and Material Transport Failure Modes for Embankment Dams’, BOR, 2000, which has been updated to incorporate lessons learnt and take account of the SPT.

It should be appreciated that there is often insufficient statistical information and models for calculating probability do not exist, making it necessary to judge the likelihood of various events or conditions. The likelihood of each branch, and the number of branches, are highly significant to the overall probability achieved; for example, the overall probability of an eight branch event tree where six branches have a low probability is effectively the low numbers adopted on the branches to the power of the number of branches with low numbers.

Chapter 8 of the BPTM (Reclamation 2009) provides a good overview of the issues, including the following key issues.

- People's ability to judge the likelihood does not extend far out either side of the probability scale (that is, to more than a couple of orders of magnitude for example lower than 0.01).
- Most people are overconfident of their judgement (a self test to estimate 10% and 90% confidence limits of 10 items is included in the chapter).
- It is best if estimates can be made in a team setting, where discussion can draw out (and record) arguments for particular values being adopted.

This guide adopts the BPTM probability mapping scheme (Table 12.2), with ranges up to 0.01 plus one more increment to differentiate between degrees of 'very unlikely'.

Table 12.2 Quantitative descriptor mapping scheme

Descriptor	Assigned probability	Comment
Very likely	0.99	As Chapter 8 of BPTM (Reclamation 2009) except that 'Virtually certain (0.999)' not used as no difference in outcome from 0.99.
Likely	0.9	
Neutral	0.5	
Unlikely	0.1	
Very unlikely	0.05	Additional descriptor as used on the Loyne dam (Mason 2010).
Highly unlikely	0.01	
Virtually impossible	0.001	

12.1.2 Step 2b – Likelihood of failure due to external threats for embankment dams

This is an assessment of critical external load conditions for the dam, namely

- the load type, magnitude and likelihood of occurrence
- system response to that load, and thus load required to cause failure (release of the reservoir)

This is similar to Tier 2 but other threats and failure modes may be considered (Table 12.3), and both the estimation of load and the system response are likely to be evaluated using more accurate (and thus resource intensive) methods. This section provides commentary on the most common issues in carrying out a Tier 3 assessment; additional information is given in Chapter 18.

Table 12.3 Other potential external threats (note 1)

Threat (in alphabetical order)	Failure mode	Suggested reference for initial estimates of probability of	
		Threat (load)	System response
Aircraft strike	Impact on dam creating breach	Thompson et al. (2001)	Bespoke analysis
Human error	Inadvertently fully open spillway gates	-	Unlikely to full release of reservoir
Snow/ice	Generate load onto dam	USACE (2002) and ICOLD (1996)	Instability of concrete dams
Terrorism	Deliberate breach by excavation, or explosion in crest, or within gallery	Use expert judgement in dialogue with company security staff	Bespoke analysis
Vandalism	Unlikely to lead directly to failure of dam (release of reservoir).	As Terrorism	
Wind	a) Tree uprooted b) Structural damage close to crest during extreme conditions c) Slope protection damage (in addition to analysis of wave overtopping)	Average wind speed of top 10 windiest locations in the UK = 14.3 mph Highest recorded gust at low level = 140 mph (Met Office 2013. See http://www.metoffice.gov.uk/public/weather/climate-extremes/)	a) Bespoke analysis to assess likelihood of displacement of root ball leading to breach b) International Levee Handbook (Note 2) for wave run-up and overtopping c) HR Wallingford, (1996) for slab slope protection

Notes: [1] Modified from Brown and Gosden (2004).
[2] Project coordinated by CIRIA, publication expected 2013 (www.leveehandbook.net).

Fragility curves and system response to load

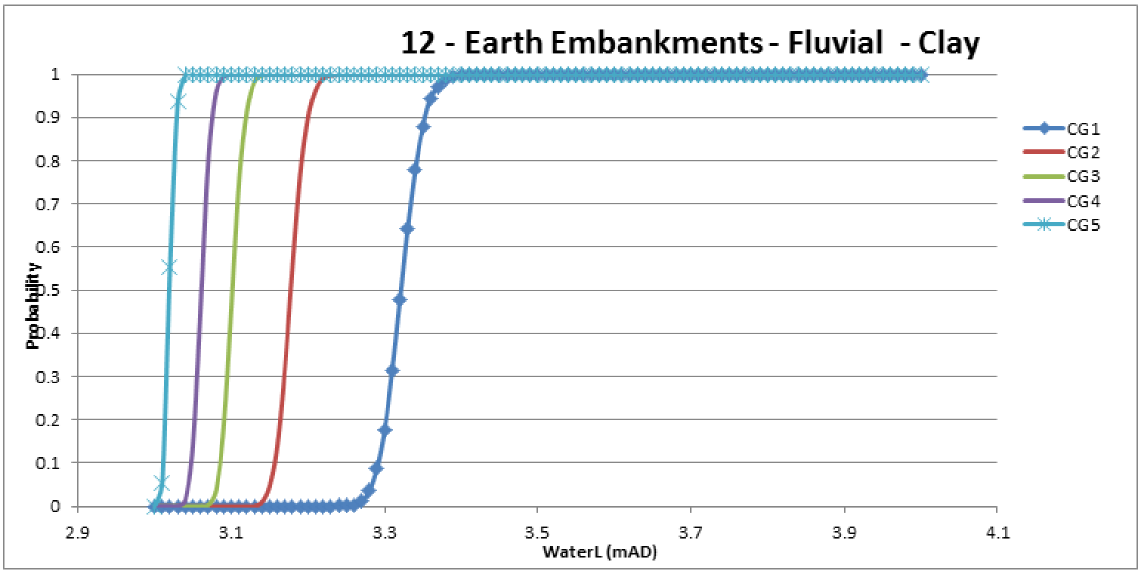
At entry level for Tier 3, rather than calculating a single point condition when failure will occur, points either side of this condition are also calculated, giving the lower bound load condition for which failure will definitely not occur and an upper bound condition for which failure will definitely have occurred.

For more advanced Tier 3 analysis the system response is defined as a (fragility) curve of probability of failure against loading. An example is shown in Figure 12.3.

Generation of such curves normally requires Monte Carlo analysis with the gradient of the curve reflecting the range of variability of the input parameters. The shape of the upper part of the curve also provides information on whether failure (release of the reservoir) is likely to be brittle (sudden with little warning) or ductile (be gradual over a period of time, giving increased warning).

Production and generation of fragility curves is covered in sources such as USACE (2010) and Simm et al. (2008). Although a more complex analysis than limiting equilibrium expressed in terms of safety factor, the use of fragility curves is more useful to risk analysis as it provides information on extremes of behaviour. This in turn is more useful in terms of probability. Note that the same safety factor can be equivalent to a wide range of probability depending on the range of input parameters – see section 21.4.2.

Figure 12.3 Example of fragility curve



Magnitude of flood load

For a Tier 3 analysis greater certainty as to potential flood flow conditions is required and hence use of the methodologies given in the *Flood Studies Report* (FSR) (NERC 1975) and *Flood Estimation Handbook* (FEH) (CEH 1999) is necessary. These provide a more reliable estimate of return period flood conditions at the reservoir and may either be used to refine the accuracy of the Tier 2 analysis (that is as an 'entry level' Tier 3 analysis) or to support a more extensive fault tree based analysis of performance.

Since introduction of the FEH, concerns have been expressed as to the reliability of extreme flood prediction. While studies to resolve this were initiated some years ago, a clear solution has yet to be published. In the meantime, guidance issued by the Reservoir Safety Working Group on behalf of Defra in March 2004 remains appropriate. Key recommendations were:

- The FEH should not be used for the assessment of 1 in 10,000 year return period rainfall. The design rainfall values provided by Volume 2 of the FSR should continue to be adopted until new guidance is provided.
- For 1 in 1,000 year return period rainfall, assessments should be undertaken for both the FEH and FSR methodologies. The more extreme of these design rainfalls should be used for flood assessment.
- The FEH should be used for the assessment of the 1 in 193-year return period rainfall (suitable for the estimation of the 1 in 150-year return period flood event).

The full text from this guidance document, *Defra – Revised Guidance for Panel Engineers on FEH – 2004*, can be found in the list of historical guidance documents on the reservoir safety page of the British Dam Society website (www.britishdams.org).

Wave height/run-up

Where a more detailed analysis of wave conditions than the method outlined in Tier 2 is required, guidance should be sought from the EurOtop assessment manual.

The EurOtop project was completed in 2007 (EurOtop, 2007) and provides the latest guidance on methods for wave analysis. The applicability of these methods for use in reservoirs is discussed in Allsop et al. (2010).

The European Overtopping Manual can be accessed from the EurOtop website (www.overtopping-manual.com), which also offers a range of online tools that allow the user to rapidly estimate potential wave conditions at a structure.

12.1.3 Step 2 – Likelihood of failure for other dams types

For a Tier 3 analysis of dam types other than embankment dams the event tree method described in Tier 2 is suggested but with inclusion of, where reasonable, a wider range of variable permutations in the event tree analysis – the aim being to reduce uncertainties that may be inherent from the conservative approach adopted at Tier 2 (see Box 8.10). More information (for example, from measurements of parameters) may allow non-conservative assumptions to be made for example about initiation and the progression of failure modes. In addition, upper ranges in the value of properties may be included which, together with conservative lower values, can provide a ‘envelope of’ probability rather than a single point conservative assumption. Such analysis can help to identify which ‘elements’ of the event tree are contributing most to the risk of potential failure and help to target interventions more effectively.

12.2 Step 2c – Dam break and flood routing

For a Tier 3 assessment of dam break, a range of numerical models may be used to predict the breach hydrograph and the subsequent flood routing.

For breach prediction through earthen embankments, use of simple rapid breach prediction models such as AREBA (van Damme et al. 2011) is appropriate for an entry level of analysis. For a more detailed assessment, use of predictive models such as HR BREACH and WinDAM would be more appropriate.

For predicting breach through thin concrete, mass gravity and masonry dams, use of indicative breach guidelines given in CIRIA Report C542 (Hewlett et al. 2000), combined with use of a flow model to predict discharge through the breach, is appropriate as an initial form of analysis. Where a more detailed assessment is required, numerical analysis of the structure should be undertaken.

For service reservoirs, judgement combined with numerical modelling of the flow will provide an estimation of potential flood outflow.

When considering cascade failure, use of breach models such as AREBA, HR BREACH or WinDAM will allow direct prediction of whether an earthen structure will breach under overflowing conditions, or not. When linked with a dynamic 2D flow model, the combined breach and flow simulation will predict how the entire cascade might propagate. When considering cascade failure with other types of dam it will require judgement to determine when failure will occur unless more detailed structural analyses are undertaken.

Predicting flood conditions downstream should be undertaken using rigorous 2D flow models suitable for simulating extreme and rapidly varying flow conditions. These models should not be simplified flow models as these are not designed to cope with the extreme and rapidly varying conditions found during a dam break. Consideration should be given to possible event trains (that is, secondary dams, blockages and failures) and sensitivity to modelling parameters.

See Chapter 19 for more detailed information on dam break and flood routing.

13 Tier 3 – Steps 2d–f Consequences of failure and risk

Step 2	Risk Analysis	Step 2d	Consequence analysis
		Step 2e	Determine level of risk
		Step 2f	Review outputs
		Optional	Estimate range of uncertainty

13.1 Step 2d – Consequences of failure

This sub-section implements the scoping of consequence scenarios of dam failure for a Tier 3 analysis given in Step 1b.

There are many sub-categories into which inundation impacts could be discretised. A selection of these is listed in Table 20.6. Table 20.12 provides suggested sources of further information for these receptors which could be included in the assessment if required. More extensive and detailed guidance on calculating flood damages is provided in the Flood Hazard Research Centre's 'Multi-Coloured Handbook' (FHRC 2010).

As a minimum it is suggested that the following categories be included:

- people (including life risk)
- economic activity
- the environment
- cultural heritage

13.1.1 Risk to people

Population at risk (PAR)

Assess the population at risk for each of the consequence scenarios defined in Step 1. At entry level for Tier 3 it is normal to consider groups of properties or people, with similar levels of exposure duration and hazard to life, rather than considering individual properties. See section 20.2.1 for further guidance.

Highest individual risk (HIR)

Determine HIR by applying a method selected from the semi-empirical equation approach (Graham 1999) for fatality rates to simulation approaches such as LSM, LIFESim, and a simplified version of LIFESim in HEC FIA. (See section 20.3.2.)

Average societal life loss (ASLL)

At entry level the average societal life loss is similar to a Tier 2 analysis, except in using a GIS database to provide more detailed calculations of population groups and risk. The user can also decide whether to include an estimate of the likely number of injuries and impact on human health; guidance is given in section 20.2.

13.1.2 Economic activity

Residential and non-residential/commercial property

At entry level this is similar to a Tier 2 analysis, except that a GIS (or similar) database is utilised to provide more detailed calculations of groups of property. Details of databases on property types available at the time of preparing this guide are given in Table 20.12.

Transport and critical infrastructure

Costs of impacts to transport and critical infrastructure systems be considered at Tier 3 and where incurred should be calculated if possible. Refer to methods and tables in FHRC (2010) for further guidance.

Agriculture

Costs of impacts to agriculture should be considered at Tier 3 and where incurred should be calculated if possible. Refer to methods and tables in FHRC (2010) for guidance.

The environment

Identify designated areas of conservation as per Tier 1 and 2, but extract the size of area affected (in ha) from a GIS (or similar) database.

13.1.4 Cultural heritage

Identify cultural heritage assets at risk as per Tiers 1 and 2.

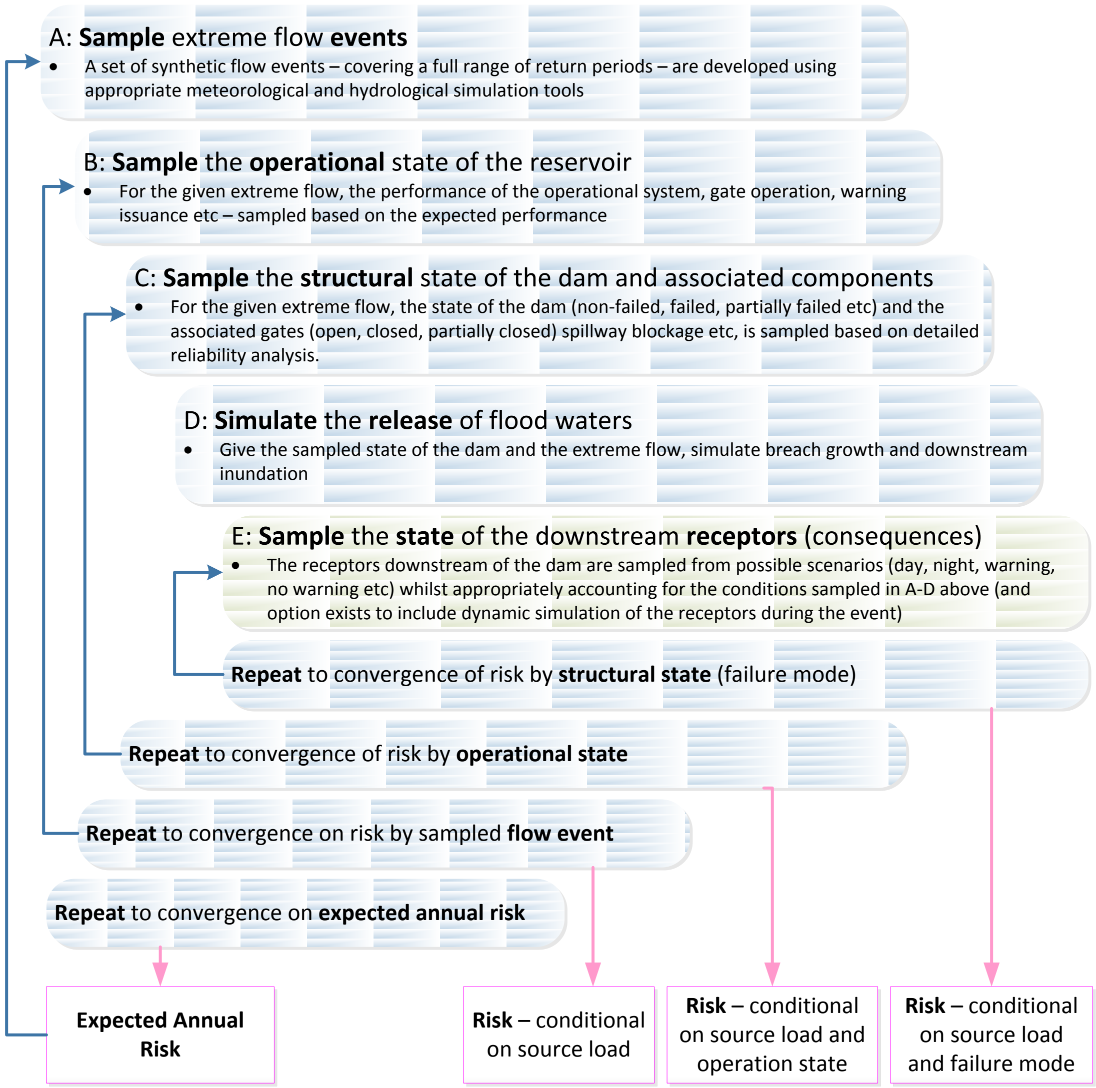
13.2 Steps 2e and 2f – Estimation of level of risk and review

13.2.1 Determining the level of risk

The approach outlined in Tier 2 can be readily expanded as the description of the hazard and consequence scenarios becomes more detailed; in this case the approach remains fundamentally a Tier 2 approach. It is likely therefore that a Tier 2 approach will be sufficient for the majority of situations (in terms of the integration of the hazard and consequence scenarios in risk). Moving beyond Tier 2, the use of pre-defined scenarios is replaced with the use of a simulation approach.

As an entry level in Tier 3, an event-based simulation based on sampling from distributions provides a practical first step towards a comprehensive analysis. Within an event-based simulation the state of the reservoir risk system is based on sampling from the probability distributions associated with each component of the reservoir risk system (taking account of any correlations). Events continued to be sampled until acceptable convergence of the variance in the risk estimates is achieved (including the expected annual damages as well as the expected consequences associated with specific flood events). Acceptable convergence will be site-specific but is likely to be based on factors that drive the decision-making at that particular site (including levels of tolerable risk, expected annual risk, benefits and costs, and so on). An event-based simulation approach will necessarily be supported by bespoke software (for example, based around existing approaches such as DAMRAE and RASP (Sayers and Meadowcroft 2005). A basic flow chart showing the key steps in this approach is provided in Figure 13.1.

Figure 13.1 Process to estimate the level of risk



Tier 3 provides an opportunity to simulate the hazard and consequence system. This type of approach is bespoke and likely to be reserved for the highest risk situations.

These types of approaches offer the potential for much more in-depth consideration of the risks. For example, the relationship between external load and the chance of specific failure modes occurring can provide a 'risk profile' of the dam and the potential consequences of failure (see Figure 13.2).

13.2.2 Displaying the results

Table 13.1 shows an example combining probabilities and consequences to calculate and record risk. This table may be produced considering each failure mode and also as a summary of all failure modes. By doing this not only the overall risk posed by the dam may be seen but also the contributions from the various failure modes, thus highlighting where action might be most effective for reducing risks.

Table 13.1 Example summary of the risks (probability and consequences) at Tier 3

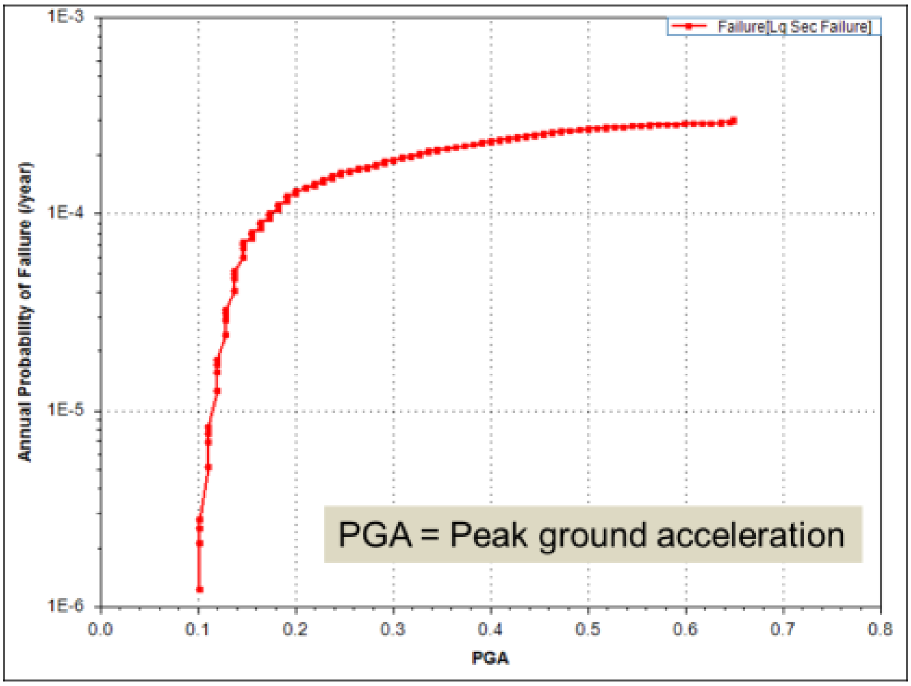
Inundation extent	**Consequences**					**Probability**		**Risk**		
				Qunaitifed		Annual probability of		Annual expected risk conditional on		
For example, partial or full release	Scenario No.	Likelihood of consequence scanrio	Metric	Lower bound	Upper bound	Lower bound	Upper bound	Mean (arthmetric)	Lower bound (LL)	Upper bound (UU)
1	1	0.5	LLOL	1	2	0.0009988	0.0009988	0.0015	0.0010	0.0020
			Residential damage	1000	1500			1.2485	1.00	1.50
			Commerical damages	2000	3000			2.4970	2.00	3.00
	2	0.5	LLOL	2	4			0.0030	0.0020	0.0040
			Residential damage	2000	3000			2.4970	2.00	3.00
			Commerical damages	4000	6000			4.9940	4.00	5.99
	All consequence scenarios	1	LLOL					0.0022	0.0015	0.0030
			Residential damage					1.8728	1.4982	2.2473
			Commerical damages					3.7455	2.9964	4.4946
2	1	0.5	LLOL	1	2	0.0009988	0.0009988	0.0015	0.0010	0.0020
			Residential damage	1000	1500			1.2485	1.00	1.50
			Commerical damages	2000	3000			2.4970	2.00	3.00
	2	0.5	LLOL	2	4			0.0030	0.0020	0.0040
			Residential damage	2000	3000			2.4970	2.00	3.00
			Commerical damages	4000	6000			4.9940	4.00	5.99
	All consequence scenarios	1	LLOL					0.0022	0.0015	0.0030
			Residential damage					1.8728	1.4982	2.2473
			Commerical damages					3.7455	2.9964	4.4946
All inundation scenarios	LLOL							0.0045	0.0030	0.0060
	Residential damage							3.7455	2.9964	4.4946
	Commerical damages							7.4910	5.9928	8.9892

Figure 13.2 Example risk profile showing the expected risk related to the driving threat and the annual expected risk

Threat	Conditional probability of failure (given threat)		Consequences (averaged across all scenarios - LLOL)		Risk - conditional on threat		
	Lower bound	Upper bound					
Return period (years)	Assumed dependent (Eq. 21.1)	Assumed independent (Eq. 21.2)	Lower bound	Upper bound	Mean	Lower bound	Upper bound
10	0.000666	0.000680	1.500000	3.000000	0.001520	0.001000	0.002039
100	0.006648	0.006780			0.015156	0.009972	0.020341
1000	0.064807	0.066053			0.147685	0.097210	0.198160
10000	0.504607	0.511186			0.766779	0.000000	1.533559
					Annual expected risk	**0.001120**	**0.003050**

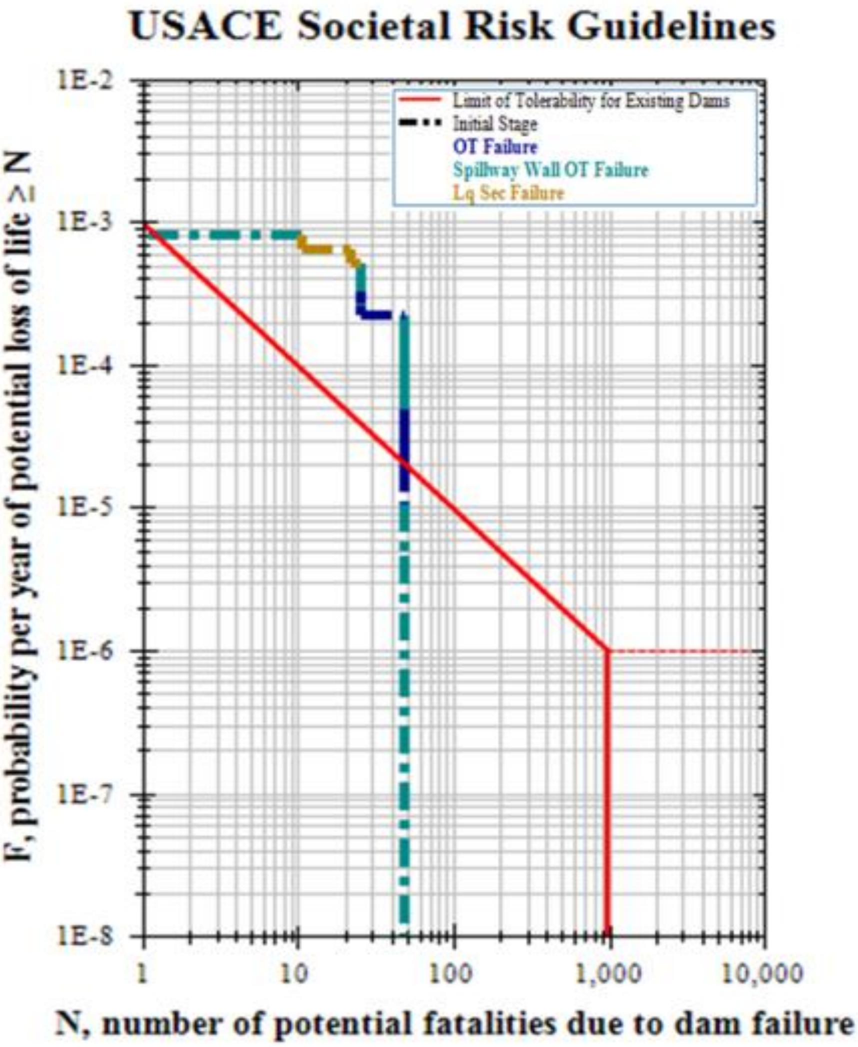
Notes: The example given is for LLOL.

Figure 13.3 Example output from a Tier 3 analysis – DAMRAE – probability of failure due to varying load severity



Equally more detailed insight into societal and individual risks are possible, as shown in Figure 13.4, which depicts a graph from an application of DAMRAE.

Figure 13.4 Example output from a Tier 3 analysis – examples from DAMRAE – societal risk



13.2.3 Step 2f – Review outputs

It is important to review the outputs of the consequence analysis.

- Are all important receptors accounted for in the assessment?
- Do the results look credible/realistic? (see Section 15.2.4, and additionaly for concrete dams Section 17.5.5)
- Does the analysis need revisiting or refining with better information?
- Where could it be improved?

Conduct a critical review of the outputs, considering whether it can be carried forward or whether any of the aspects in Step 2 should be refined. This could, for example, include the need for more accurate data, or moving to a higher tier (and hence complexity) of analysis. Issues to consider include:

- What governs the overall consequence scenario?
- What governs the total probability?

- Do the risk levels look about right?
- Where are the gaps and what do I need to know more about?

13.2.4 Optional – Estimate range of uncertainty

Uncertainty handling and estimation is central to any Tier 3 analysis. There are various options for incorporating uncertainty and Figure 13.1 reflects a sampling approach where values are drawn from distributions of parameters. Supporting structured sensitivity analysis (such as variance-based sensitivity methods) provides valuable support to help identify those uncertainties that contribute most to uncertainty in the estimates of probability, consequence and risk. Implementing such approaches must be considered on a case-by-case basis and requires expert input.

14 Tier 3 – Step 3 Risk evaluation

The following steps are recommended for risk evaluation in support of a Tier 3 risk assessment. These steps should be conducted for the reservoir in its existing state and for any risk reduction options that are considered. A traffic light system is suggested for summarising the risk evaluation outcomes. The steps address accepted good practice, tolerability of risk, and any additional decision bases identified in Step 1a – Scoping.

Step 3	Risk Evaluation	Step 3a	Review tolerability of risk assessment
		Step 3b	Review options to reduce risk
		Step 3c	Proportionality
		Step 3d	Other considerations
		Step 3e	Review and make recommendations

14.1 Step 3a – Review tolerability of risk

The **tolerability of risk** evaluation should include the following two parts:

- **Highest individual risk** (IR). Limit of 1 in 10,000 per year. If the maximum estimated individual risk is less than the IR limit value a ‘Yes’ outcome is assigned, or an apparent ‘A Yes’ outcome is assigned if the risk assessment needs to be improved to satisfy confidence and defensibility requirements of the owner/undertaker or the inspecting engineer. If the maximum estimated individual risk exceeds the IR limit value, then either a ‘No’ or an ‘A No’ outcome is assigned depending on whether or not confidence in the risk assessment satisfies the requirements of the owner/undertaker and the inspecting engineer.
- **Societal risk** (SR). Limit as shown by the sloping line on the F-N chart in Figure 5.3. If the risk plotted on the F-N chart to represent the estimated societal risk is less than the SR limit line over its entire range then a ‘Yes’ outcome is assigned, or an apparent outcome is assigned as ‘A Yes’ if confidence in the risk assessment needs to be improved to satisfy requirements of the owner/undertaker and the inspecting engineer. If the line plotted on the F-N chart to represent the estimated societal risk is above the SR limit line at any point over its entire range then either a ‘No’ or an ‘A No’ outcome is assigned depending on whether or not confidence in the risk assessment satisfies the requirements of the owner/undertaker and the inspecting engineer.

14.2 Step 3b – Review options to reduce risk

The identification of potential options to reduce risk, and quantification of the costs and benefits of these options is similar to the assessment for Tier 2, except being in more detail.

14.3 Step 3c – Proportionality

The quantitative evaluation of whether risk reduction measures to achieve ALARP are proportional is similar to the assessment described in Tier 2, except that sensitivity analysis would normally be carried out.

14.4 Steps 3d and 3e – Other considerations, review and recommendations

14.4.1 Step 3d – Other considerations

As well as comparing the benefits and costs of potential risk reduction measures, the risk assessment should consider the following questions for any practical options that can be identified to further reduce the risk.

1) **Have either the IR or SR evaluations in Step 3b resulted in a 'No' or 'A No' outcome?**

Compare the level of risk with the limit guidelines. If either the individual risk or societal risk evaluations in Step 3b resulted in a 'No' or an 'A No' outcome, then ALARP has not be demonstrated unless there are extraordinary circumstances preventing these limit guidelines being met by any practical risk reduction measures.

2) **Does the risk assessment satisfy the confidence and defensibility requirements of the owner or undertaker and any other stakeholders?**

As appropriate, ensure that any societal concerns are adequately addressed. Stakeholders, including those who would be affected by dam failure or dam repairs should be consulted and their concerns addressed. The outcomes of this evaluation can be indicated by 'Yes' or 'No', or 'A Yes' or 'A No' if the risk assessment does not satisfy the confidence and defensibility requirements of the owner or undertaker, and any other stakeholders.

3) **Have all the risk guidelines identified in the pre-assessment (see Chapter 2) been adequately addressed?**

Additional risk criteria (as identified in the pre-assessment) can be listed and if appropriate the outcomes of evaluating them can be indicated using 'Yes', 'A Yes', 'A No', or 'No'.

4) **Does the dam meet published engineering standards for the UK?**

Confirm whether the dam has been assessed against published standards such as floods and seismic design, and if so did it meet the published standards, or are there outstanding deficiencies? (Refer to Section 2.4.6.)

5) **Have any deficiencies identified in previous studies been addressed?**

Confirm whether the risk assessment has taken into consideration previously identified deficiencies (that are unresolved), and whether or not these have been addressed in the risk assessment.

14.4.2 Step 3e – Review and make recommendations

Where possible, compare the outputs of the risk assessment with similar dams.

Bring together both the risk analysis (including the quantitative evaluation of risk reduction measures (Step 3c) and considerations of other factors (Step 3d) to make a decision recommendation with an accompanying justification (refer to the list of decision issues identified in the scoping step).

PART TWO – Supporting information

15 Basis of the tiered approach

15.1 Introduction

This section supplements Chapter 4 of Volume 1 in describing the basis of the tiered system.

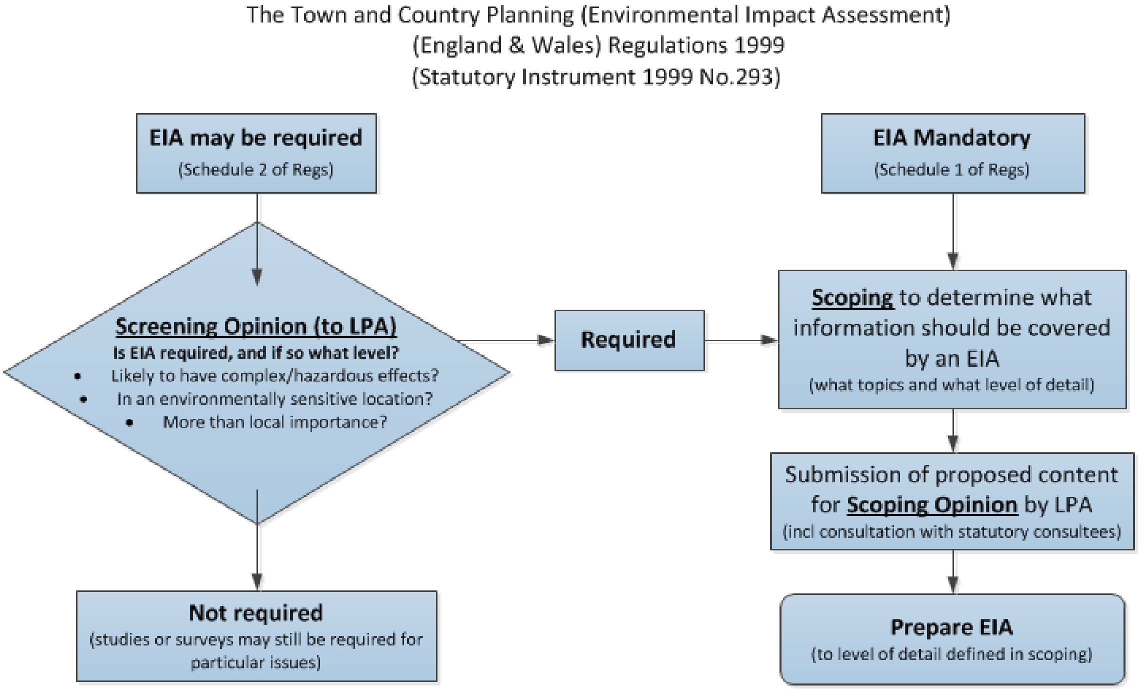
As society and governments realise that it is not possible to eliminate all risk (for example from ill health, extreme weather and other natural catastrophes), there is a shift towards a risk-based approach to management of risk. Examples include the publication of *Reducing Risk Protecting People* (R2P2) (HSE 2001) and the culture shift evident in recent years in organisations such as Defra and the Environment Agency – typified by changes in official terminology such as from 'flood protection' to 'flood risk management'.

In seeking to understand such a diverse behaviour of the natural world, risk analysts and practitioners have recognised the importance of 'tiered' approaches as a way of:

- managing the complexity in many risk issues
- taking analysis to a level of detail appropriate to the decision
- carrying out analysis consistent with the level of data/information available

Implicit in a risk-based approach is a tiered approach, where simplistic analysis is carried out first to screen out significant risk, followed by more detailed assessment of the significant risks. Such an approach to risk assessment and analysis is not new, a precedent is the Town and Country Planning (Environmental Impact Assessment) (England and Wales) Regulations 1999 (Statutory Instrument 1999 No. 293) which provides a tired approach to Environmental Impact Assessment (EIA) as shown in Figure 15.1.

Figure 15.1 Tiered approach to environmental risk assessment



In a well-designed risk analysis system, there should be consistency between these different levels of analysis, even though the issues considered may well be different. As the tier of the assessment descends the risk assessment methodologies should become more specific to a particular problem/decision as the level of detail increases.

The way risk is expressed depends on the tier. Risk screening may simply be a matter of identifying whether a particular risk could arise (for example, whether there is a possibility of harm as a result of the hazard and the vulnerability of the likely receptor), whereas at the detailed level outcomes may be expressed in probabilistic terms and include assessment of options for measures to reduce risk. However, at each stage of the risk assessment process the conceptual approach to understanding and assessing risk should be the same and will typically follow a structured and well-used path (Table 15.1). Where detailed analysis is unwarranted any or all of the stages may be conducted at various levels of detail or approximation. This applies to the quantity of data required to conduct the analysis, the sophistication of the analysis methods and the significance of the decision being taken. The level of detail chosen is then reflected in the accuracy and level of confidence placed on the analysis results (Defra and Environment Agency, 2002).

Table 15.1 provides a summary of the tiered approach adopted in this guide. Tier 1 is the simplest approach, comprising a qualitative assessment of risk. Tier 2 introduces basic quantitative analysis and Tier 3 more detailed quantitative methods.

Table 15.1 Tiered analysis

Tier	Type of risk assessment	Description
1	Qualitative	Ranking of potential failure modes, and order of magnitude likelihood and consequences using a descriptive risk matrix. Optional sensitivity analysis.
2	Simplified quantitative	Threshold analysis using hand calculations that is with basic calculator. Optional sensitivity analysis.
3	Detailed quantitative	Range of levels. Include system response curves, with range of initiating events (threats) using computer software for risk calculations. Uncertainty dealt with by formal sensitivity to full uncertainty analysis.

Failure modes identification (FMI) underlies each of the tiered approaches. A basic approach to FMI is recommended for Tier 1, with a more detailed approach adopted for Tiers 2 and 3.

An explanation of the differences between the tiers, and guidance on how to choose which level of assessment to use, is provided in Chapter 4 of Volume 1 of this guide.

15.2 Basis of a tiered set of tools

15.2.1 Methodological balance and key simplifications in lower tiers

As outlined in Table 15.1 the tiered approach requires different levels of assessment methodology and analysis from the simplified to the more complex as required by the tier. There was a significant amount of discussion during the development of this guide over the appropriate balance between

(a) simplicity of use

(b) need for transparency in the process (so non-experts can do the calculations themselves, and thus gain confidence in risk assessment output)

(c) accuracy of output

The solution that was eventually incorporated in the Guide is summarised Table 15.2. The risks of the accuracy of the output being overestimated are reduced by recommending that users complete an assessment of confidence in the components of the risk assessment, as part of Step 2f.

Table 15.2 Compromise between accuracy of output and simplicity incorporated in this guide

Element of risk assessment	Compromise	Practical drawbacks
Failure modes identification	Tier 1 structured as core threats,* with user to identify additional failure mode, rather than brainstorming from blank page	Increased risk of overlooking critical failure mode
Partitioning of load domain	Tier 1 and 2 both consider single 'dam critical' load rather than curves of load vs. probability, which are then integrated with curves for system response	May overlook critical response at intermediate load. Position of step may not be best estimate.
Reservoir level vs. time	Assume normally full.	Although this is valid for many UK reservoirs (for example, amenity lakes) it will be conservative where the lake is well below TWL for significant proportions of the year.
System response	Tier 1 and 2 both consider single (step) response (probability), rather than two (or multiple) point fragility curve.	As above
Consequence scenarios	Tier 1 and 2 limited to one and two scenarios respectively.	Less accurate (probably conservative)
Tools to identify and quantify number of receptors	Tier 1 and 2 allow use of published 1:25,000 scale map, rather than requiring computer based assessment.	Less accurate identification and quantification of receptors
Presentation of risk output	Tier 1 and 2 limited to total probability, rather than individual failure modes (and uncertainty bounds on those estimates)	Need to drill down into individual failure mode to understand the critical threats

Notes: *Analysis undertaken when developing the Interim Guide and other portfolio risk analysis in UK concluded that the threat to UK reservoirs from earthquakes is not significant in comparison with other threats. Earthquakes have therefore not been included as a core threat in Tier 1 or Tier 2 as they are generally not significant (unless there is a liquefiable foundation). Where mining activity has been common place in the area of the dam, consideration to the effects of subsidence on the dam may be undertaken. However, such analyses are likely to be very site specific and specialist and would warrant a Tier 3 analysis. The susceptibility of all dams and reservoirs to acts of vandalism or terrorism should be considered as part of routine reservoir safety management and are not considered further in this guidance.

15.2.2 Tier 1 – Devising a qualitative system to rank risk

Although Tier 1 is qualitative, the verbal descriptors of likelihood and consequences should be derived from quantitative values to ensure consistency of scoring between different threats, and ranking of different consequences of failure. The link to likelihood of failure is shown in Table 15.3, and consequences and the resultant risk matrix in Table 15.4. Both are based on Chapter 31 of Reclamation's Best Practice Trailing Manual (Reclamation 2010b) except in adding extreme likelihood of failure to the likelihood of failure axis.

Table 15.3 Quantitative values associated with Tier 1 likelihood of failure

Indication of range of likelihood of release of reservoir, expressed as annual chance			Example of similar probabilities (every day events) – as Appendix 4 to R2P2 (HSE 2001)
Note 1	High (1 in)	Low (1 in)	
Extreme		100	Zone 3 in UK flood and coastal erosion risk management (FCERM) flood risk ('high risk' used in FCERM)
Very high	100	1,000	Zone 2 in UK FCERM flood risk (PPS 25 uses the term 'low risk' when < 1 in 1000)
High	1,000	10,000	Annual risk of death due to accidents and external causes (averaged over whole UK population – 1 in 4,000)
Moderate	10,000	100,000	Annual risk of death due to road accidents (averaged over whole UK population 1 in 17,000)
Low	100,000	1,000,000	Annual risk of death from industrial accidents to employees in service industry (1 in 333,000)
Very low (remote)	<1,000,000		

Table 15.4 Quantitative values associated with Tier 1 likelihood of failure and risk matrix

	Magnitude of consequence				
	0	1	2	3	4
	Reclamation BPTM Chapter 31				
	Level 0	Level 1	Level 2	Level 3	Level 4
	Indicative average societal life loss (equivalent in Tier 2)				
Likelihood	<0.01	<0.1	<1	<10	>10
Extreme	Levels of risk shown as follows: Risk matrix = coloured gradation from green to red for increasing risk Tolerability - by the words for tolerability				
Very high					
High					
Moderate					
Low					
Very low					

15.2.3 Purpose of risk assessment

Preparation for the risk identification process, including establishing the context and objectives is described in Chapter 2 of this volume of the guide. It is important to appreciate that there are several different reasons why a risk assessment may be carried out such as:

(a) Risk of release of reservoir (dam safety)

(b) Risk of damage to the dam

(c) Operational issues such as on-going wave erosion which need remediation and thus provision made in capital budgets for asset management

This Guide is concerned primarily with (a). Where (c) is an important part of the purpose of the risk assessment, appropriate adjustment should be made to the techniques described in this guide

15.2.4 Review and validation of output

It is important that the output from application of the guide to an individual (or group of) dam(s) is reviewed critically by an engineer experienced in dam engineering. Published results of use for QRA include those listed in Table 15.5. There are many other papers on principles, but this list is limited to those where quantitative output is published for UK dams.

In addition, Figures 15.2 to 15.4 comprise selected extracts from a joint Defra and Environment Agency research project (FD2641) completed in 2010 (unpub). The sample of dams from which these plots were prepared consist of a number of groups of dams (350 total) on which detailed quantitative risk assessment had been carried out. Not every dam has both probability and consequences of failure, such that some dams only have one component of risk estimation. The sample is biased is that it is under-representative of small dams with, for example, median height and reservoir capacity being 17m and 600,000m^3 compared with a median for UK of 8m and 130,000m^3 respectively. However, it does provide an indication of the range of typical QRA output for UK dams. Further detail on the sample is given in Interim Paper 3 of the Defra research project. Information on historical failure rates for concrete dams is given in Section 17.5.5.

Table 15.5 Sources of published quantitative results of QRA on UK dams which may assist in validating output from this guide

Aspect	Published quantitative results
Physical attributes	• Skinner, H., 2000. The use of historical data in assessing the risks posed by embankment dams. *Dams & Reservoirs*, 10(1), 9-12. • Tedd, P., Skinner, H.D. and Charles, J.A., 2000. Developments in the British national dams database. In: *Dams 2000*, Proceedings of 11th Biennial British Dam Society Conference, 14–17 June 2000, Bath, pp. 181-189. Thomas Telford, London.
Internal threats	• Brown, A.J. and Tedd, P., 2003. The annual probability of a dam safety incident at an embankment dam, based on historical data. *International Journal of Hydropower & Dams,* 10(2), 122-126. • Eddleston, M. and Carter, I.C., 2006. Comparison of methods to determine the probability of failure due to internal erosion in embankment dams. In: *Improvements in Reservoir Construction,*

Aspect	Published quantitative results
	Operation and Maintenance, Proceedings 14th Conference of British Dam Society, Durham, 6–9 September 2006, Paper 51. Thomas Telford, London. • Gosden, J.D. and Dutton, D., 2008. Quantitative risk assessment in practice. In: *Improvements in Reservoir Construction, Operation and Maintenance*, Proceedings 14th Conference of British Dam Society, Durham, 6–9 September 2006, Paper 41. Thomas Telford, London.
Risk including consequences	• KBR Pilot study included in 'Floods and Reservoir safety integration'. (Brown and Root, 2002) (*Includes in Chapter 7 and Volume 3 a pilot study of application of draft method on ten dams).* • Hughes, A.K. and Gardiner, K.D., 2004. Portfolio risk assessment in the UK: a perspective. In: *Long-term Benefits and Performances of Dams*, ed. H. Hewlett, Proceedings British Dam Society 13th Biennial Conference, Canterbury, 22–26 June 2004. Thomas Telford, London. • Brown, A.J., Yarwood, G., King, S.J. and Gosden, J.D., 2008. *Application of the Interim Guide to Quantitative Risk Assessment across multiple dam owners by multiple Jacobs offices*, Proceedings British Dam Society 15th Biennial Conference, Warwick, 10–13 September 2008, Paper 13, pp. 65-79. • Defra and Environment Agency, 2010a. *Scoping the Process for Determining Acceptable Levels of Risk in Reservoir Design.* Project FD2641. Task 3: Assessment of reservoir characteristics affecting reservoir safety risks [key graphs reproduced here]

The following plots from the joint Defra and Environment Agency research project, 'Scoping the Process for Determining Acceptable Levels of Risk in Reservoir Design' (FD2641), can be used to assist you to validate the results of the risk assessment.

Figure 15.2 Indicative distribution of population at risk and average societal life loss (Defra and Environment Agency 2010. unpub)

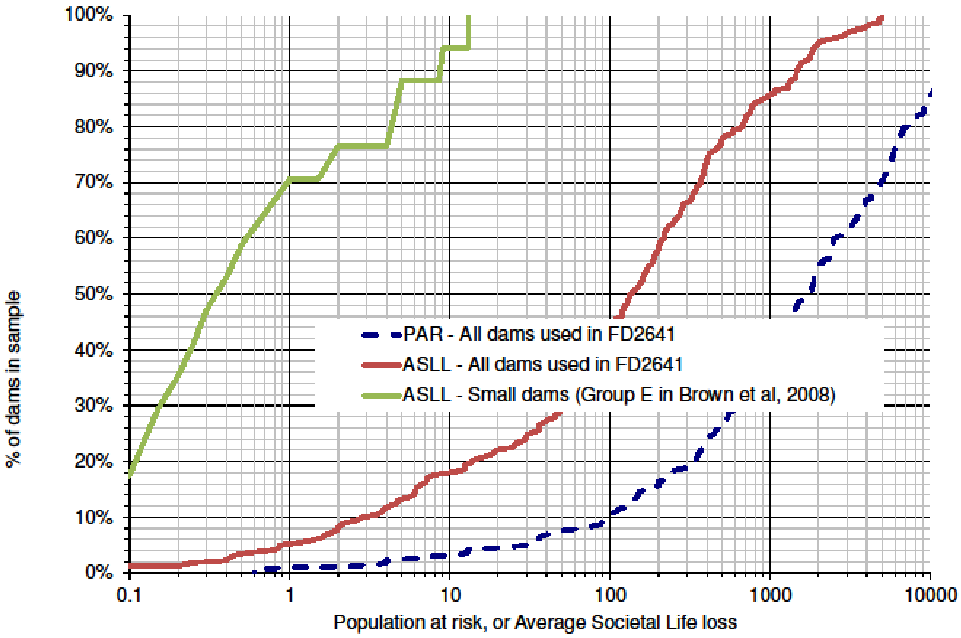


Figure 15.3 Indicative cumulative distribution for UK embankment dams of overall probability of failure (Defra and Environment Agency. unpub)

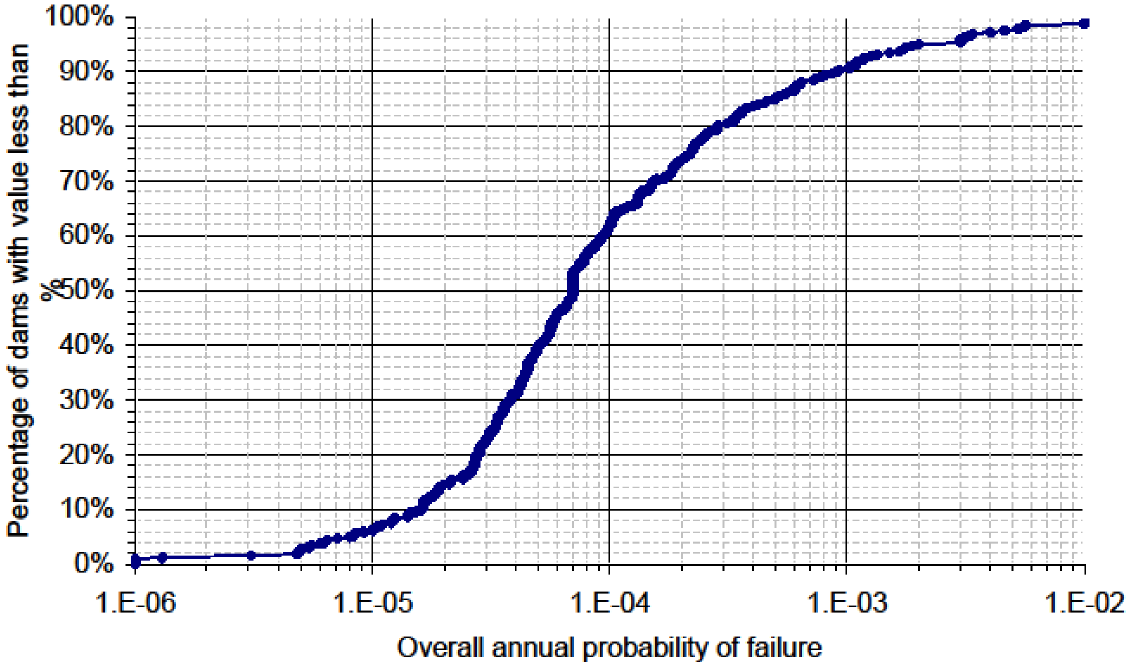
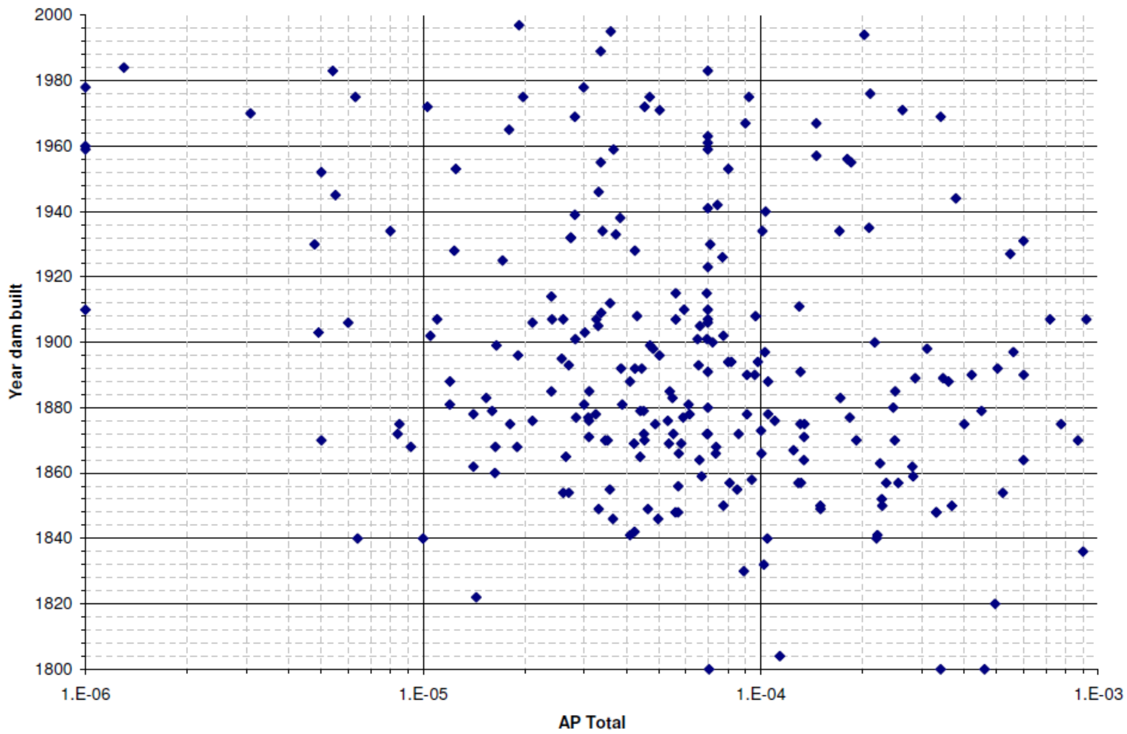


Figure 15.4 Indicative distribution for UK dams of overall probability of failure vs. age (Defra and Environment Agency. unpub)



15.3 Data quality and adequacy of data

Quantitative information on dam condition, construction details, geotechnical parameters and so on is often incomplete or scarce. This, coupled with the complex interactions between load and response (structural deterioration, breaching, human response and so on), means that there is uncertainty associated with the assessment of performance.

To reduce uncertainties through improved data collection is often expensive and the usefulness of improved information is limited by our ability to predict behaviour. It is therefore important to consider the requirement for further data collection for risk assessment.

The assessment itself can be used to assist in the identification of the requirement to improve data coverage or quality if uncertainty is included as part of the analysis. Poor quality or coverage of data generally increases uncertainty in the assessment and conversely better data coverage and quality will decrease uncertainty in the analysis results.

A risk assessment should therefore recommend, where appropriate, further data collection where high uncertainty in the results exists and the consequences of failure are significant. Where uncertainty is more marginal, or consequence of dam failure are likely to be low then a balance needs to be struck between cost and reduction in uncertainty. The level to which uncertainty needs to be reduced has to be considered on a case-by-case basis as it depends largely upon the level of risk exposure that dam owner is willing to except or that which a regulator may determine as ‘adequate’. See section 21.4.4 on further application of uncertainty and sensitivity analysis.

16 Guidance on failure mode identification – Step 1a

This section describes the purpose and overall approach to failure modes identification, the role of evidence, a systematic approach to identification of failure modes, screening and classifying potential failure modes as credible and significant, some available resources, and the important topic of describing failure modes and documenting the identification process.

16.1 Introduction

Failure mode identification is one of the most important steps in a risk assessment. It ensures that all significant sources of risk, and their causes and potential causes are identified and included in the risk analysis (BS ISO 31000:2009). Techniques of risk identification are summarised in Table A.1 of BS EN 31010:2010 and can include elements of Failure Modes and Effects Analysis (FMEA) (BS EN 60812:2006).

This section describes the basis of the guidance provided in Part 1 on the steps that are required to provide reliable failure mode identification.

Comment on the identification (scoping) of consequence assessment, and scoping the risk analysis is given in Chapters 20 and 21 respectively.

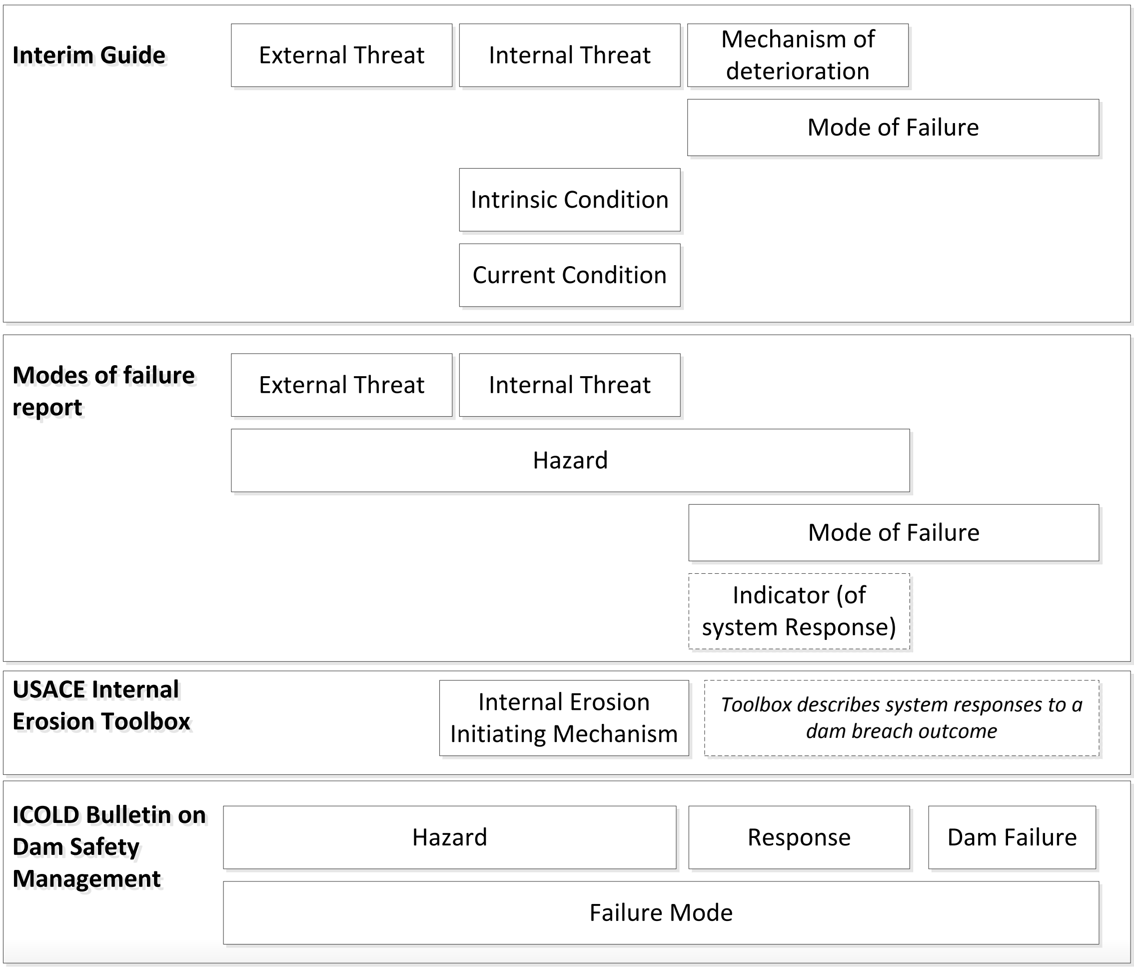
16.2 Key issues and concepts

16.2.1 Definitions

A significant challenge to utilising UK and overseas work on modes of failure is the inconsistent use of terminology. Figure 16.1 indicates the relationships between the terminology used in this guide, and the terminologies used in three reference documents shown in Figure 16.1 and ICOLD Bulletin 154 Dam Safety Management (ICOLD 2011). One of the difficulties with definitions is that what may be an indicator for one mode of failure (for example, increased seepage due to internal erosion) may be the initiating threat to a separate mode of failure (slope instability).

The definitions used in this guide are shown in the Glossary and in a simplified form in Table 3.1.

Figure 16.1 Relationship between terminologies in the reference documents for failure modes identification



16.2.2 The role of evidence

It is important to recognise that to perform a comprehensive failure modes identification one should not rely on a purely observational approach. Such an approach focuses on visual evidence of performance (surveillance) or indicators based on monitoring and measurements. Rather, one should also consider what current conditions could exist even though there is no evidence for them, perhaps because there is currently no means of detecting the condition.

In addition to considering the current condition of the reservoir system, one should also consider the intrinsic condition of the reservoir system, for example the absence of internal filters, based on the available evidence, such as as-built drawings or boring logs. However, one should also consider what undesirable intrinsic conditions cannot be ruled out because it would not be unreasonable that they may exist in a particular dam considering factors such as contemporary design and construction practice, or likely site geological and geomorphological conditions. In other words, the absence of evidence for an undesirable condition is not evidence of the absence of the condition.

So in contrast to a scientific hypothesis, which is only proven if there is sufficient evidence to support the hypothesis, or a legal standard of proof, which may require that the evidence for a verdict is greater that the evidence against the verdict, in matters of reservoir safety one should consider the possibility that some failure modes may exist, even in the absence of evidence, that is, they may be credible even if not evident. This positive demonstration of safety is the standard that we would expect from the inspection of a plane before we fly on it, and the public should expect no less for safety of a reservoir.

An important resource for identifying potential failure modes is the literature on case histories of dam incidents and dam failures (Environment Agency 2011a).

Where the evidence is insufficient to demonstrate that a particular failure mode exists but the failure mode cannot be ruled out, it should be included in the risk assessment. However, where there is uncertainty as to either the existence or the severity of a failure mode, the effects of these uncertainties on the estimated risk should always be understood. In addition, practical ways of reducing these knowledge uncertainties should be identified and implemented where the uncertainties could result in the reservoir posing an unacceptable risk.

16.3 Basis of tiered methodology for failure modes identification (FMI)

16.3.1 Description and condition of dam

It is important that the form of construction and condition of the various elements of the dam are understood when carrying out the failure mode identification. This aspect has therefore been made common to all tiers by including guidance in Chapter 2 (preparation for risk assessment) as follows.

- Section 2.4.1 shows the various aspects that should be differentiated in any description of a dam.
- A checklist of the elements of different types of dams is given as Table 16.1.
- A checklist of data that are normally available is given in section 2.4.
- Reference is made to any up-to-date engineering assessment for the dam (see section 2.4.6).

Table 16.1 Reference lists for description of dam

Checklist for elements of dam for description of Intrinsic condition			Current condition in relation to dam failure (release of reservoir)
Embankment	Gravity		
Dam section	Concrete/masonry dams	Service reservoirs	
Geometry			
Source/ type of fill materials used			Movement (settlement)
Crest wall	Vertical contraction joints forming blocks	Roof construction	

Wave protection	Formwork/ facing	Roof/ wall joints	
Internal drainage/ filters	Pressure relief wells	Floor construction	Drainage flows
Abutment mitre – detailing		Floor/ wall joints	
		Underdrain	
		Back of wall drain	
Foundation			
Type of soil/ rock			
Treatment of formation			
Grout curtain?			
Spillway		(overflow for SR)	
Weir level			Maximum historic water level
Walls			
Piers/ bridge across spillway			
Gates			
Culverts/ tunnels through dam			
Tunnel (in rock) or culvert (in fill)?			
Materials and size			
Weepholes?			
Interface with fill			
Pipework and valves			
Size/ type for each of			
Draw off			
Bottom outlet/ scour			
Access for operation			
Reservoir			
Stability of rim		Level of alarms (normally Hi, HiHi, Low, LowLow)	
By wash/ inflows			
Surveillance			
Instrumentation Emergency planning/ operation			

16.3.2 Escalation of FMI process between tiers

The FMI process is effectively the same between all three tiers. What varies between tiers is:

- the time spent and level of detail rather than the basic process
- the level of evidence needed to decide a threat is not significant

- detail of the record of the FMI evaluation process

The number of ‘core’ failure modes suggested to be considered is shown in Tables 3.1 and 7.3. However, any other credible and significant failure modes identified for a specific dam should be added to this list.

Table 16.2 Comparison of detail of failure mode identification with tier

Tier	Core failure modes suggested to be considered
1 Qualitative	Limited to modest list shown in Table 3.1
2 Basic quantitative	More comprehensive checklist in Table 7.2
3 Detailed quantitative	Detailed assessment, including typically a single A4 sheet to record assessment of each candidate failure mode

It is important to realise that the FMI process may identify complex failure modes for which the tier does not provide a means of estimating a likelihood of failure. In such cases, it would normally be appropriate to move to the next tier and associated more sophisticated method of estimating probability of failure.

16.4 Supporting information

16.4.1 Aids to the FMI process

FMI is an important process which demands critical thinking by the assessor. Thought was given to including checklist in the guide, but it was considered preferable to indicate ranges of threats in Table 7.3 and include in Tables 16.3 and 16.4 a schedule of available checklists for embankment and concrete dams respectively. Table 16.5 lists some significant failure modes for service reservoirs.

Table 16.3 Checklists that assist with FMI of embankment dams

Engineering guide	Reference	Type of Information	Comments/Location of checklist in reference
Interim Guide	Brown and Gosden (2004)	FMI for dams	Refer to 'Event trains' (one provided for each threat)
Small embankment reservoirs (CIRIA Report 161)	Kennard et al. (1996a)	Surveillance checklist for indicators of potential failure modes	Appendix Q
The safety of concrete and masonry dams (CIRIA Report 148)	Kennard et al. (1996b)		Chapter 7 and Appendix 4
Valves, pipework and associated equipment in dams (CIRIA Report 1997)	Reader et al. (1997)		Section 6.1 and 8.1 cover monitoring
Modes of dam failure and monitoring and measuring techniques	Environment Agency (2011b)	Checklist of failure modes and indicators	Table A.2 provides a matrix of 11 modes of failure and 48 hazards, Table A.3 provides matrix of hazards and indicators Main text provides sketch ad description of each hazard and mode of failure
Management of flood embankments (R&D Report FD 2411)	Defra and Environment Agency (2007a)		Chapter B2.2 describes typical failure mechanisms
The safety of embankment dams	Johnson et al. (1999)	Surveillance	Table 8 and Appendix D
Internal erosion toolbox	USACE (2009)	Schedule of FM	Lists 28 number internal erosion initiating mechanisms
Potential failure mode identification, description and screening (Chapter 1, BPTM)	Reclamation (2010c)	Process for FMI	Although no checklist is given, it provides an excellent summary of the FMI process.
Lessons from historical dam incidents	Environment Agency (2011a)	Historical incidents	
Checklist in this guide – Tier 1	Section 3.1 of this guide	Threats, failure modes and breach types	Table 3.2
Checklist in this guide – Tier 2	Section 7.1 of this guide	Matrix of combinations of initiating threats and progression (Failure Modes)	Table 7.2

Table 16.4 Checklists which would assist with FMI of concrete dams

Engineering guide	Reference	Type of Information	Comments/location of checklist in reference
The safety of concrete and masonry dams (CIRIA Report 148)	Kennard et al. (1996b)	Surveillance checklist for indicators of potential failure modes	Chapter 7 and Appendix 4
Valves, pipework and associated equipment in dams (CIRIA Report 1997)	Reader et al. (1997)		Section 6.1 and 8.1 cover monitoring

Table 16.5 Examples of some significant failure modes for service reservoirs

Type of service reservoir	Threat	Progression	Comment
Perimeter wall dependent on external fill for stability	Overpumping	Roof locally displaced, Fill eroded by overtopping	
	Pipe burst in perimeter fill	Jet of water from burst erodes fill, loss of support to reservoir wall leading to structural collapse	
All	Foundation deterioration	Weakening of foundation (many possible causes) leads to foundation instability	*Lessons from Historical Dam Incidents* (Environment Agency 2011, p. 98) includes an example of a sudden major leak from Mill Hill service reservoirs in Durham.

Notes [1] CIRIA Report 138 (Johnson et al. 1995) provides useful information on design and detailing of service reservoirs
[2] Other features of service reservoirs that increase vulnerability to failure (release of reservoir): (a) thin, short panels with no continuity of steel, no shear key across expansion joints (applies to all forms of construction for example RC, mass concrete gravity and so on); and (b) significant cross fall on ground so that base of reservoir is at about original ground level at the corner of the reservoir where original ground is lowest.

16.4.2 Process for describing failure modes

The following extract is from Chapter 1 of Reclamation's Best Practices Training Manual (Reclamation 2010c) on potential failure modes analysis (PFMA):[11]

> 'It is important to put scale drawings or sketches up on the wall, and sketch the potential failure modes during the discussions. The potential failure modes must be described fully, from initiation to breach and uncontrolled reservoir release. There are three parts to the description:
>
> - **The initiator.** For example, this could include increases in reservoir due to flooding (perhaps exacerbated by a debris-plugged spillway),

[11] Underlined text has been added to the original text.

strong earthquake ground shaking, malfunction of a gate or equipment, deterioration, an increase in uplift, or a decrease in strength.
- **Failure Progression.** This includes the step-by-step mechanisms that could lead to the breach or uncontrolled release of the reservoir. The location(s) where the failure is most likely to occur should be also be highlighted. For example, this might include the path through which materials will be transported in a piping situation, the location of overtopping in a flood, or anticipated failure surfaces in a sliding situation.
- **Breach mechanism.** The method and expected magnitude of the breach or uncontrolled release of the reservoir is also part of the description. This would include how rapid and how large the expected breach would be, and the breach mechanism. For example, the ultimate breach from a piping failure mechanism adjacent to an outlet conduit might result from progressive sloughing and unravelling of the downstream slope as a result of flows undercutting and eroding the toe of the dam, until the reservoir is breached at which point rapid erosion of the embankment remnant ensues, cutting a breach to the base of the conduit.

The reasons for completely describing the potential failure modes are: (1) to ensure the team has a common understanding for the follow-on discussions, (2) to ensure that someone picking the report up well into the future will have a clear understanding of what the team was thinking, and (3) to enable development of an event tree or other means of estimating risks, if warranted.'

In addition to describing the initial-progression-breach aspects of the failure mode, it may also be of interest to summarise the potential consequences of the failure mode occurring. This can be useful in scoping the estimation of consequences to adequately capture all significant consequences.

Some reference lists in UK publications are summarised in Table 16.1. These should not be used as 'checklists' that limit the failure modes considered to those mentioned in these lists. They can be used to stimulate the lateral thinking that is useful in identifying potential failure modes that maybe unique to a particular reservoir system.

Other valuable UK references on potential failure modes include the following reports:

- *Lessons from Historical Dam Incidents* (Environment Agency 2011a)
- *Management of Flood Embankments*, Chapter 2 (Defra and Environment Agency 2007a)
- *International Levee Handbook*, Chapter 3 (CIRIA 2013, see www.leveehandbook.net)

16.4.3 Examples of describing failure modes

Some examples of descriptions of failure modes adapted from Chapter 1 of the BPTM Reclamation (2010c) are shown in Boxes 16.1 to 16.3.

Box 16.1 Example failure mode description (Reclamation 2010c)

Unedited (insufficient detail): Piping through the embankment
Edited: Piping of the embankment core initiates at the gravel transition interface. The core material is carried through the gravel transition zone and rockfill shell material, and into the waste berm at the toe of the dam. Backward erosion occurs until a 'pipe' forms through the core to the upstream gravel transition beneath the reservoir level. At that point, flow through the 'pipe' increases, eroding the core material until the gravel transition and upstream shell collapse into the void, forming a sinkhole in the upstream face. Continued increase in flow erodes and enlarges the 'pipe' until the crest collapses into the void and the embankment is breached. Erosion continues to the base of the dam, about elevation 2960.

<u>More-likely and less-likely factors</u>
Adverse or 'More Likely' factors:

- The gravel transition zones do not meet modern 'no erosion' filter criteria relative to the core base soil.
- The gravel transition zone may be internally unstable, leading to erosion of the finer fraction through the coarser fraction and even worse filter compatibility with the core.
- The reservoir has never filled to the top of joint use; it has only been within 9ft of this level; most dam failures occur at reservoir levels reached for the first time, which may occur here for a 1 in 50 to 1 in 100-year snowpack.
- The core can sustain a roof or pipe; the material was well compacted (to 100 percent of laboratory maximum), and contains some plasticity (average PI~11).
- There is a seepage gradient from the core into the downstream gravel transition zone, as evidenced by the hydraulic piezometers installed during original construction (and since abandoned).

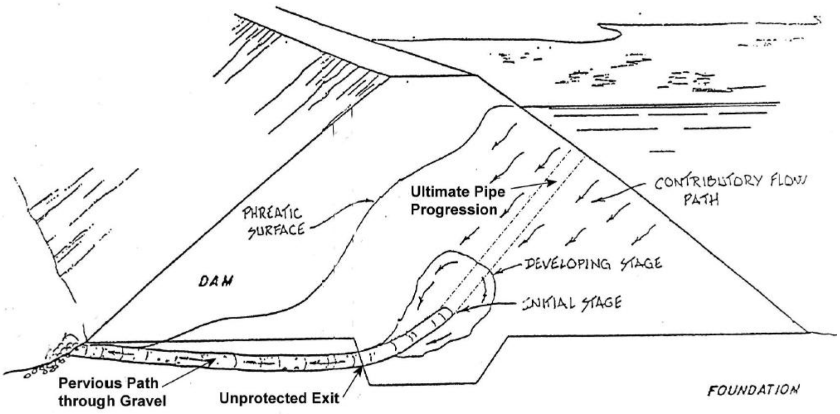
Favorable or 'Less Likely' factors:

- Very little seepage is seen downstream, the weir at the downstream toe, which captures most of the seepage through the dam, records about 10 gal/min at high reservoir when there is no preceding precipitation, indicating the core is impermeable; this level of flow is unlikely to initiate erosion.
- The core material is well compacted (to 100% of laboratory maximum) and has some plasticity (average PI~11), both of which reduce its susceptibility to erosion.
- There are no known or suspected defects in the core where erosion could initiate; benches in the excavation profile that could cause cracking are above the joint use elevation.
- If erosion of the core initiates, the gravel transition zone may plug off before complete breach occurs, according to the criteria for 'some erosion' or 'excessive erosion' by Foster and Fell (2001).

<u>Effect:</u> If this potential failure mode were to initiate, it would be difficult to detect due to the coarse rockfill shell and waste berm downstream which would hide the seepage. The downstream weir is affected by precipitation that often masks the true seepage. Therefore, the failure mode could be well developed and in progress by the time it is detected. Once the core of the dam is breached to the reservoir, breach could occur within a few hours.
<u>Consequences:</u> If the East Dam were to breach by this mechanism, at risk would be two county roads, several farmhouses, two bridges, a branch railway line, a motorway, a petrol station, an aggregate plant, a lumber mill, a transmission line, and the town of Tannerton at about 30 miles downstream. There is little recreation activity downstream of the dam. The total population at risk is estimated at about 90.

Box 16.2 Example failure mode description (Reclamation 2012)

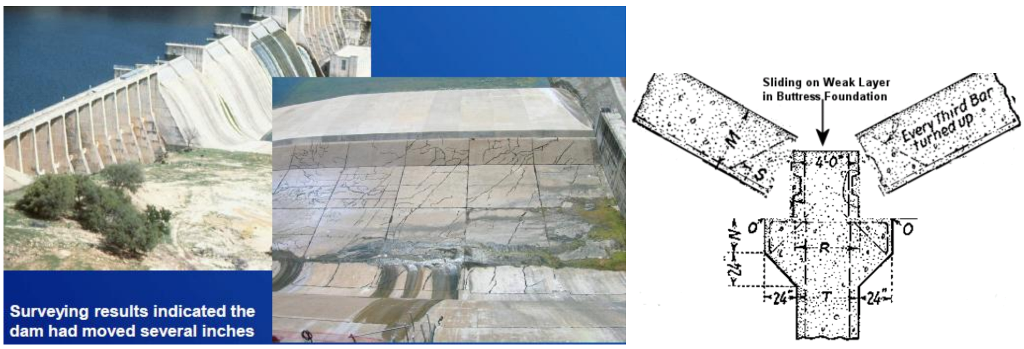
Unedited (insufficient detail): Piping from the embankment into the foundation



Edited: During a period of high reservoir elevation, piping of the embankment core initiates at the gravel foundation interface in the shallow cutoff trench near Station 2+35 (where problems with the sheet pile and sinkhole occurred). Material might or might not exit at the toe of the dam. Backward erosion occurs until a 'pipe' forms through the core exiting upstream below the reservoir level. Rapid erosion enlargement of the pipe occurs until the crest of the dam collapses into the void, and the dam erodes down to the rock foundation.

Box 16.3 Example FM description (Reclamation 2012)

Unedited (insufficient detail): Sliding of the concrete dam foundation



Edited: As a result of high reservoir levels and (1) a continuing increase in uplift pressure on the old shale layer slide plane, or (2) a decrease in shearing resistance due to gradual creep on the slide plane, sliding of the buttresses initiates. Major differential movement between two buttresses takes place causing the deck slabs to be unseated from their simply-supported condition on the corbels. Breaching failure of the concrete dam through two bays quickly results followed by failure of adjacent buttresses due to lateral water load.

16.4.4 Seismic risk in the UK

Previous analyses have shown that the seismic risk in the UK is moderate by world standards; the *Interim Guide to Quantitative Risk Assessment* (Brown and Gosden

2004) concluded that, for embankment dams, the risk was ‘insignificant’ compared with other risks to be incorporated as a core threat. Accordingly, seismic analyses are not recommended for Tier 1 or 2 levels of analysis for embankment dams.

However, concrete dams and service reservoirs are considered more vulnerable to failure in an earthquake, and so earthquakes are included as core threats at Tiers 1 and 2.

Some possible causes for undertaking a seismic analysis of an embankment could include:

- embankment dam constructed primarily with saturated sandy soils or on saturated sandy foundations
- embankment dam in poor condition
- old embankment dam with steep slopes and the presence of loose fills
- dam includes vulnerable structures such as a high draw off tower

In this situation the user is advised to follow the assessment methodology detailed in the BRE publication, *An engineering guide to seismic risk to dams in the United Kingdom* (Charles et al. 1991’). This guide uses data on reservoir capacity, dam height, population at risk and downstream damage potential to assign a dam category (I, II or III), which leads onto one of the three classifications in Table 16.6.

Table 16.6 Dam categorisation for seismic risk

Seismic classification for embankment dam	Seismic classification for concrete dam	Description
E_a	C_a	In general no seismic safety evaluation is required.
E_b	C_b	Look for features particularly vulnerable to earthquake damage and undertake seismic analysis only if such features found.
E_c	C_c	Undertake a staged evaluation using levels of sophistication appropriate to the situation.

Notes: Source: Charles et al. (1991)

If the screening analysis results in an Eb/Cb (with particularly vulnerable features), or an Ec/Cc classification then a more detailed seismic analysis might be required. This could include a pseudostatic analysis or a dynamic finite element model analysis for example. Refer to Charles et al. (1991) and ICE (1998) for more detailed guidance.

17 Likelihood of failure due to internal threats – Step 2a

This chapter provides supporting information in respect of failure due to internal threats. It contains additional guidance and a summary of the supporting science.

17.1 Definitions

This guide defines internal threats as:

> 'pre-existing internal flaws or process which lead to deterioration of a dam sufficient to be the root cause of failure. This may lead directly to failure under constant load, or may weaken the dam to such an extent that it fails rapidly when subject to a change in external load'.

The basis of this definition is given in Table 17.1.

Support for this definition of internal threats is provided by (i) Hartford and Baecher (2004, Table 8.4), which suggests initiating events are flaws (leading to internal erosion), deterioration and mechanical or electrical failures; and (ii) FLOODsite Report T04-06-01 (Allsop et al. 2007). Sections 2.3.2 and 5.2 of the latter state:

> 'Examples of the challenges that time-dependency introduces are: the representation in the probability of failure of failure processes that are dependent upon the history of loading. …The significance of deterioration/ time dependent process combined with the relatively poor level of understanding and modelling ability means that this is an area where research needs to be increased during the coming years'.

Internal threats differ from external threats; the latter are measurable and a probability of occurrence of defined loads can be determined for them.

Table 17.1 Basis of strategy for assessment of likelihood of failure due to internal threats

	Statement	Comment
1	Failure initiated by internal threats can occur with no change in reservoir level due to: • change in moisture content (consolidation) due to the combined effects of self weight settlement, surface infiltration/ evaporation and seepage; generally with consequential changes in stress distribution within the body of the dam. • movement of soil due to progression of internal erosion • surface desiccation	**a)** The initiating event may be termed 'internal threat' or 'internal instability' and is akin to fatigue loading in bridges. **b)** Internal threats cannot be modelled by a single system response (fragility) curve, which relates probability of damage to water level. **c)** This would be a change from the FCERM approach where internal erosion is seen as a failure mode under increase in water level – see FLOODsite Report T04-06-01 (Allsop et al. 2007) Table 3.1 (Failure Mode

	Statement	Comment
	• cyclic loading of reservoir, seasonal temperature and moisture content change • anything that leads to 'sunny day' failure	1.5) and Failure Mechanism sheets Ba1.5 and so on. **d)** Although not critical for flood defence structures, the difference is important for dams which permanently retain water.
2	Stability failures can be triggered by • an external load such as water level or wave overtopping • internal threats such as desiccation leading to tension cracks along the slope, or cyclic loading leading to strength reduction	Slope instability would be defined as failure mode rather than 'root cause'.
3	It is important to differentiate the 'root cause' from the 'effect' of a mechanism of deterioration/mode of failure – see Table 7.2.	This is because data on historical events generally relate to effects, while what is required for prediction of failure is 'root cause'.
4	Index rating systems should differentiate current condition from intrinsic condition.	This is to aid post-incident investigations to understand the root cause probability of failure and how this is affected by maintenance.
Internal erosion as a failure mode		
5	For movement of soils to occur within the body of a dam at least two of the three factors of grain size, hydraulic load and stress state must be susceptible to internal erosion (see section 4.1).	This implies that initiation can occur without hydraulic load that is a crack can open up. Particle detachment would only occur once water load was applied.
6	There are four different types of internal erosion – concentrated leakage, piping, contact erosion and backward erosion.	As ICOLD Bulletin on internal erosion (ICOLD 2012)
7	Failure due to internal erosion can also occur in response to increase in water load, the vulnerability depending on whether the dam has been weakened by the processes described above.	These could be modelled by system response (fragility) curves, although a family of curves would be necessary to model pre-existing damage.

17.2 Key issues and concepts

This section builds on the definition of internal threats given above to describe some of the key issues with reference to current science.

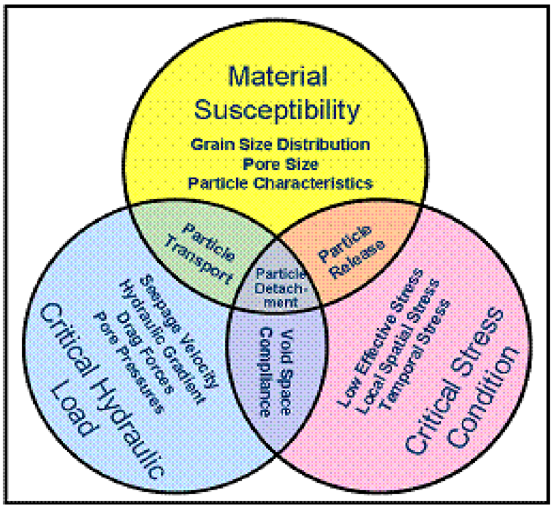
17.2.1 Internal erosion

Despite recent research, the physical mechanisms controlling initiation and the rate of development of internal threats are still not fully understood, and for internal erosion are believed to be controlled by all of the three elements shown in Figure 17.1.

Systems that provide a means to estimate the likelihood of failure due to internal threats therefore have to be based on judgements as to:

- physical processes which could lead to failure
- any surface expression which would be expected
- any evidence that such surface features are present
- any monitoring of parameters within the dam

Figure 17.1 Venn diagram illustrating interaction of geometric, hydraulic and mechanical susceptibilities of soils to initiation of internal erosion



Source: Garner and Fannin (2010)

Issues that need to be addressed in any estimation of the likelihood of failure due to internal threats include:

- the practical observation with UK dams that leaks can initiate with no change in reservoir level
- the fact that it is sometimes not easy to differentiate cause from effect
- UK experience shows that frequent surveillance inspections of dams is extremely effective in preventing dam failure, as it identifies structural problems early on and gives time to take action to prevent failure

Other contributing factors which affect the likelihood of failure of dams due to internal erosion include:

- variability in soil properties
- singularities (plus construction details)
- local groundwater
- water chemistry
- mineralogical changes in soil properties with time
- seasonal changes in temperature

Some of these relate to the original construction and thus risk of ‘flaws’, while others relate to on-going loading and environment of the dam.

17.2.2 Deterioration of reinforced concrete structures

The design life of reinforced concrete structures is typically 100 years, so in say 200 years with no maintenance, it would be expected that widespread carbonation (or other deterioration) would occur necessitating either major refurbishment works or reconstruction.

There are a variety of published methods for predicting the service life of reinforced concrete structures such as ‘WAR’ (Conroy et al. 2011) and ‘BRET digest’ (Quillin 2001). It is assumed that the lifespan of such assets will be managed proactively, including slit trenches to allow inspection and testing of buried surfaces at appropriate point in asset life (for example, 100 years?) and remedial measures, such that the probability of structural failure is reduced to a tolerably low level.

17.2.3. Importance of surveillance

A QRA screening assessment of the probability of failures using historic performance is based on the assumption that the existing programme of surveillance, monitoring and maintenance continues, such that the probability of failure due to internal threats relates to:

- undetected defects or deterioration
- potential impacts of current deterioration (for example, seepage), assuming that if it worsens it will be mitigated/ remedied by maintenance

Reference should also be made to published engineering guides, which include both checklists for surveillance and information on typical UK dams. Sources of such checklists are given in Table 17.2.

Table 17.2 Example checklists for surveillance and information

Engineering guide	Reference	Checklist for surveillance
The safety of embankment dams	Johnston et al. (1999)	Table 8 and Appendix D
Small embankment reservoirs (CIRIA Report 161)	Kennard (1996a)	Appendix Q
Valves, pipework and associated equipment in dams (CIRIA Report 170)	Reader et al. (1997)	Sections 6.1 and 8.1 cover monitoring
Concrete and masonry dams (CIRIA Report 148)	Kennard et al. (1996b)	
Lessons from historical dam Incidents	Environment Agency (2011a)	

17.2.4. Foundation failure of concrete gravity structures

Foundation failure is the most common cause of failure of concrete dams (various data sources are summarised in section 17.5.5), with causes including (ICOLD 1993):

- increasing seepage washing out fines in joints

- cyclic loading leading due to reservoir operation/ seasonal effects weakening the foundation
- blockage of foundation drainage leading to increase in uplift
- deterioration of water stops and so on leading to increasing leakage from service reservoirs

17.3 Basis of tiered set of tools for internal threats to embankment dams

17.3.1 Objectives

The system adopted for this guide is intended to provide a tiered approach of effort, with consequential improving accuracy of the best estimate for more sophisticated methods. It is recognised that the methods in the three tiers may give different estimates and this is simply a reflection of uncertainty in estimates of the probability of failure due to internal threats, and different methods (as described in the preceding section) giving different estimates (for example, Tier 1 is a qualitative methodology).

The desirable attributes for a system for estimation of likelihood of failure should include the features summarised in Table 17.3.

Table 17.3 Target attributes of system to estimate likelihood of failure due to internal threats

	Attribute	Comment
1	Likelihood of failure (LoF) of poor condition dams (99th percentile) reflects probability of failure given a serious incident	In UK typically 1 in 70 (Brown & Root 2002, Table 6.4)
2	Median LoF matches observed average failure rate from internal stability	5×10^{-5} in UK 1975-2000 (Brown and Tedd 2003)
3	LoF of best current condition dams (1st percentile) has a value sufficiently low that it is (a) credible and (b) means that a well-built modern dam poses a tolerable risk even where extreme consequences (LLOL > 1,000) that is annual chance less than 10^{-6} to be tolerable, and 10^{-8} to be acceptable.	Value in Interim Guide (Brown and Gosden 2004) for best intrinsic condition reduced by factor of 100 for this guide to 10^{-9} (that is, probability) which is similar to that which would be applicable to failure due to floods of reservoir with full wave freeboard allowance and good condition grass. Then subdivided into different failure modes. Value for poor Intrinsic condition but good current condition taken as Interim Guide (that is, 1,000 times worse than above).
4	Reproducibility in terms of different engineers achieving the same estimate of probability when using the same system on the same dam.	Guidance updated to include Brown and Peters (2007) and other refinements.
5	Consistency of output between tiers.	
6	Clear how system deals with upgrades/ retrofitting of filters, and change in likelihood of failure is reasonable.	

17.3.2 Basis of Tier 1 methodology for internal threats

The Tier 1 (qualitative) method is a simple matrix-based system, with the adopted method being based on the key elements shown in Table 17.4, and the mapping of probability terminology to likelihood described in Chapter 6.

Table 17.4 Basis of science used in Tier 1 LoF due to internal threats

Issue	Basis of methodology used in Tier 1
Output matrix	Reclamation simplified method set out in BPTM Chapter 31 (Reclamation 2010b) except differentiating intrinsic from current condition. This is to clarify thinking and to provide realistic estimates of likelihood when a dam is clearly in structural distress.
Intrinsic condition	Adapted from Reclamation semi-quantitative method (Chapter 31)
Current condition	Simplified from the Interim Guide (Brown and Gosden 2004)

17.3.3 Basis of Tier 2 methodology for internal threats

Basis of the method

The Tier 2 method is based on the elements in Table 17.5, with the approach illustrated graphically in Figure 17.2. Alternative methodologies summarised in Table 17.6 were considered and included in the pilot study but not included in the final guide as they did not provide any benefits over the chosen system. It is also noted that the ICOLD Bulletin on internal erosion (ICOLD 2012) states:

> 'The method allows a broad categorization of the dam types into those less and more likely to fail. However the authors of the method have found that it does not allow for important details of the dam to be allowed for, and it cannot model the frequency of reservoir loading which is an important factor in estimating the likelihood of failure. They prefer to use event tree methods such as the 'piping toolbox'.

Table 17.5 Basis of science used in Tier 2

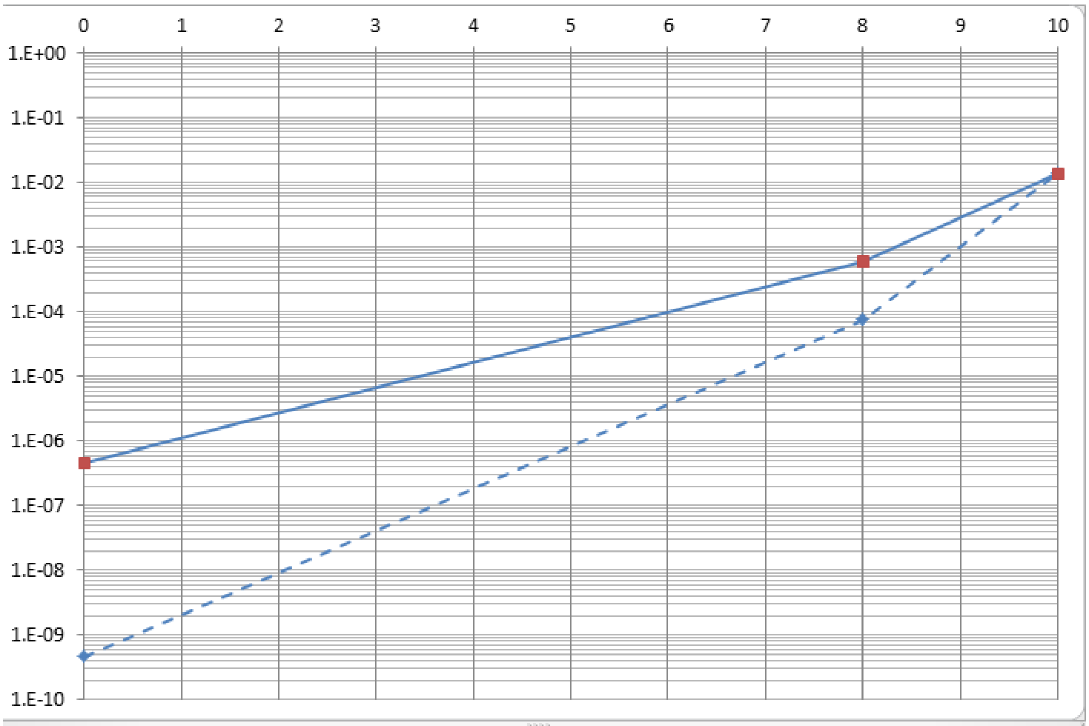
Issue	Basis of methodology used in Tier 2
Probability mapping scheme	Mapping scheme as shown in Figure 8.1, using a combination of intrinsic and current condition. The benefits of this system are that: • it provokes critical thought on the separate issues of how the dam was built and how it is performing now • the current condition score 10 can be linked to observed conditional probability of failure given that a dam is in very poor condition, and thus reflect increased probability for very poor condition dams
Estimation of probability	Poor condition dams – uses values from BRE database and extracted as part of Brown & Root research project, with high level summary reported in Brown and Tedd (2003). Full research report is available from Defra and in BDS members' area. Good condition dams (score 1) – judgement as described in row 3 of Table 17.3.
Changes from Interim Guide 2004	The cumulative scoring system presented in this guide has been modified from that in the Interim Guide (Brown and Gosden 2004) as follows. • Extend guidance on scoring condition factors as Brown and Peters (2007)

Issue	Basis of methodology used in Tier 2
	• Reduce probability of failure of anchor point 1, as Table 17.3 • Surveillance – weighting doubled and linked to likely speed of failure • Reservoir operation – no longer included as factor in scoring of current condition (for simplicity, user still free to take into account)

Table 17.6 Alternative methodologies to estimate likelihood of failure, based on historic performance

Method	Reference	Description	Comment
New South Wales	Foster et al. (1988)	Provides base probability for variety of dam types. Then vary to adjust for Intrinsic condition (with small adjustment for current condition).	Based on Australian and US data Current condition only given small weighting, so dam in poor condition (imminent failure) may not necessarily have high likelihood off failure. ICOLD Bulletin on internal erosion (ICOLD 2012) notes the authors now prefer event trees as being more reliable.
Stanford method	McCann et al. (1985)	Provides a scale of 0 to 10 for probability, driven mainly by current condition.	Based on Australian and US data Description of condition biased towards American dams

Figure 17.2 Basis of system used to infer probability of failure due to internal threats at Tier 2



17.3.4 Information in support of Tier 3 assessments

At Tier 3 there are two established (US) methods for estimating the probability of failure using event trees, namely the (SPT) (Fell et al. 2008a) and Chapter 24 of BPTM (Reclamation 2010a). However, these methods are based on international, mainly US and Australian data, such that there may sometimes be a case for making adjustments to the published US methods to account for UK dams. There is, however, no science to support such adjustments, so they would need to be engineering judgment by the user, with some comments on issues to be considered set out below.

Reclamation methodology – process to populate event tree

- **Continuation (Inadequate filter, Node 2).** This requires information on the properties of the shoulder and core in order to determine if the shoulder acts as a filter to the core. Where such information is not available then a preliminary assessment can be made on the basis of dams of a similar age and type, although to achieve confidence in the output for high risk dams, a ground investigation would normally be carried out.
- **Intervention (Node 7).** In UK there is generally a high success rate for intervention, with the BRE database showing that typically there is less than one failure for every 70 incidents. The comments in the Reclamation manual on site-specific assessment are also pertinent, such that the probability of success may vary significantly from the general rate.

17.4 Basis of tiered set of tools for internal threats to gravity structures

There are insufficient historical data on incidents at gravity structures (concrete and masonry dams and service reservoirs) to form the basis of Tier 1 and 2 assessments. The methodology included in the guide is therefore a simplified version of a methodology developed and calibrated as part of a portfolio risk assessment on these types of dam for a major UK water company.

17.5 Further references and information sources

17.5.1 Introduction

This sub-section summarises key information on construction of UK dams relevant to the assessment of the likelihood of failure due to internal threats. It includes published information on:

- typical forms of construction of UK dams
- for embankment dams
  - the range of clays used in construction of the watertight element
  - sources of data on the various geological clay strata in the UK, which are likely to have been used to construct dams situated on these strata

The following publications provide data on probability of failure rates for a range of infrastructure, including internal threats to embankment dams.

- *Failure Rate and Event Data for use within Land Use Planning Risk Assessments* (HSE 2010)
- *Deterioration Rates of Long-Life, Low Probability of Failure Assets: Project Report* (UKWIR 2011)

Symptoms of on-going deterioration

The four types of internal erosion as described in the ICOLD Bulletin (2012) are summarised in Table 4.16, together with common surface indicators.

17.5.2 UK embankment dams

Zoning of UK embankment dams

Most dams have some form of coarser fill shoulder supporting the core, such that the interaction between the shoulders and the core is crucial to whether internal erosion can develop to failure. Where the shoulders immediately adjacent to the core satisfy modern filter rules in relation to the core material, internal erosion cannot progress.

A description of the variation of characteristics of typical embankment dams in UK is given in Table 17.7. Further information on standard design and construction practice is given in Kennard (1994), Skempton (1989) and Binnie (1981). The latter includes a chapter on each of the main engineers active from around 1820 to 1900, with a list of dams mentioned in the text in an appendix. Kennard and Skempton both refer to broadly post and pre-1960 practice in UK dam design and construction.

Watertight element (core)

Moffat (2002) includes geotechnical index data on ‘puddle clays’ relevant to internal erosion; his data on Atterberg limits are reproduced in Figure 17.3.

Table 2 of Moffat’s paper shows that for two of the 32 dams for which data are provided the ‘clays’ are in fact silts, while Table 12 shows that some ‘puddle clays’ are dispersive (that is, they will de-flocculate and erode if subject to concentrated leakage). There is other evidence that some British clays may be dispersive, particularly where they form part of a sedimentary sequence of interbedded sand and clays, or are weathered from such a deposit. Although dispersive clays are not widespread, caution should be exercised as the clays used in some dams may be dispersive.

There is also evidence that at some dams the same material was used in both the shoulders and the core, the only difference being that the core was placed using a ‘puddling’ process. With time the higher initial moisture content of the core is likely to reduce, such that the dams become broadly homogenous.

Published papers on geological materials which may have been used as ‘puddle clay’ are summarised in Table 17.8.

Table 17.7 Typical features of UK zoned dams (after Moffat 2002)

Phase	Period	Max core height (m)	Core H/b	Impervious element	Cut-off provisions	Shoulders	Key dates
Early	1800-1840	25	3 to 4	Central puddle, or thick upstream (1.0–1.5m) blanket	Puddle in key trench	Random fill	1730 first use of punned clay barrier 1766 first foundation cut-off trench *c*.1795 puddle clay barrier first employed 1820 Roman cement introduced 1824 Portland cement introduced
Pennine Phase 1	1840-1865	30	3.5 to 6	Puddled in situ that is water added to clay in place in dam and puddled by boots and puddling tools/clay spades	Puddle clay in trench, as necessary	Random fill: lifts up to 1.2m; no compaction	1852 Bilberry failure 1864 Dale Dyke failure
Pennine Phase 2	1865-1880	35	3 to 5			Select fill against core (Moody zoning); lifts to 1.2m; toe drainage; compaction incidental	1870 Slip joint in culvert (Binnie 1981, p. 149) 1870 Stop using peat as support to puddle 1877 First use of cementation process to seal fissures in rock – Thomas Hawksley (Binnie 1981, p. 119 and p.153)
Pennine Phase 3	1880-1945	35	3 to 4	Puddle clay prepared ('Pugged') away from the embankments		As above but may be limited compaction	
Pennine Phase 4	1945-1960	45	3 to 5.5		Concrete in deeper trenches and grout curtain	As above but controlled compaction	
Modern	Post-1960	90	2.5 to 3	Broad rolled clay core	Rolled clay key trench and/or grouted cut-off	Engineered, with zoned compacted earthfill and/or rockfill; drainage/filters and so on	

Notes: H = maximum core height above base; b= maximum core width at base

Shoulder materials

There is no comparable published survey of the properties of shoulder materials. These will normally have been obtained by excavation in the reservoir area, and may also include spoil from tunnel and spillway excavations. The excavation methods in place at the time the dam was built should be considered and will normally dictate the maximum particle size present (for example, historic rockfills will be finer than modern rockfills excavated with large mechanical plant).

Foundation

One of the key sources of information on materials present under the shoulders of the dam is (where it exists) the longitudinal section along the cut-off excavation. In all cases, careful study should also be made of published geological information such as geological maps and sheet memoirs. Other useful references include Walters (1971), which includes a detailed account of the geology of British dams.

Figure 17.3 Range of fine grained soils used in 'puddle clay' dams (after Moffat 2002)

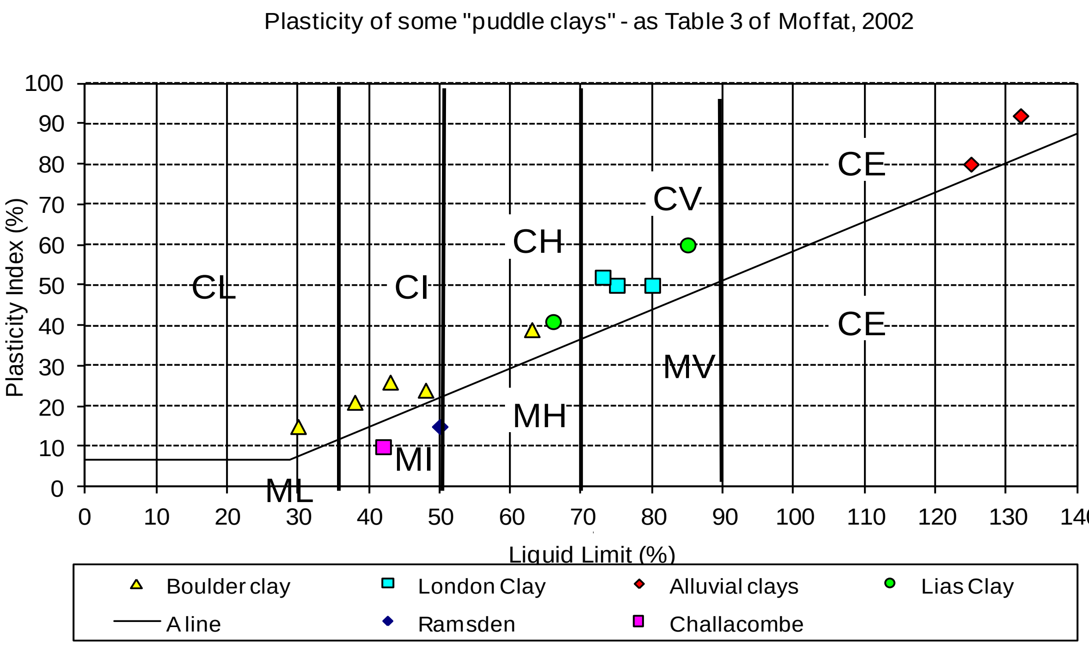


Table 17.8 Published generic information on geological materials which may have been used as 'puddle clay'

Geological origin of material	Reference
Claygate Beds and Bagshot Beds	Northmore et al. (1999)
Coal Measures	Taylor (1988)
Glacial till	Trentor (1999)
(Boulder clay)	Hughes et al. (1998)
	Kittle (1988)
Keuper Marl	Chandler (1969)
London Clay	Burnett (1992)
	Chandler and Apted (1988)
Lias Clay	Chandler (1972)
Peat	Hobbs (1986)
General	Wroth (1978)

Notes: [1] The above is not exhaustive, being limited to geological strata for which an overview of the strata properties has been identified.
[2] Although not normally used as puddle clay, peat was sometimes used on either side of a core to 'protect it' during construction

17.5.3 Service reservoirs – historical development

Historic Concrete: The Background to Appraisal (Sutherland et al. 2001) contains the following key chapters:

- Chapter 5 – summary of concrete design and practice
- Chapter 6 – changes in properties and performance of concrete
- Chapter 16 – water-retaining structures in Britain before 1920
- Chapter 17 – historic concrete in dams

Key dates from these overview papers are summarised in Table 17.9, while modern practice in design and construction of joints in concrete structures is defined in CIRIA Reports 136 (Harrison 1995) and 138 (Johnston et al. 1995).

Table 17.9 Key dates in the development of reinforced concrete in structures

Date	Author	Title/comment
1852		The (London) Metropolis Water Act first required that filtered water reservoirs be covered.
1877	Hyatt	First book on reinforced concrete
1882	Morris	First of two ICE papers on review of history of design of covered service reservoirs (references 45 and 46 in Chapter 16 of *Historic Concrete*)
1907	RIBA	First report of Committee on Reinforced Concrete to review the use of structural reinforced concrete
1915	LCC	Reinforced concrete regulations. Captured design recommendations developed over previous decade
1932	Reynolds	Reinforced concrete designers handbook (1st edition)
1934		First British code of practice
1938	ICE	Code of practice of the design and construction of reinforced concrete structures for the storage of liquids
1948		CP114
1950	Institution of	*Manual of British Water Supply Practice*. Chapter 10 covers

Date	Author	Title/comment
	Water Engineers (ed. Hobbs)	service reservoirs and water towers 4th edition published in 1969 (ed. Skeat)
1960	BSI	CP2007 – Design and construction of reinforced and pre-stressed concrete structures for the storage of water and other aqueous liquids. Includes drop test, allows 13mm over seven days
1972	BSI	CP110 – Unified CP114 to 116 (in situ, pre-stressed and precast)
1985	BSI	BS 8110 – Structural use of concrete [now withdrawn]
1987	BSI	BS 8007 – Design of concrete structures for retaining aqueous liquids [now withdrawn]
2006	BSI	BS EN 1992-3 Eurocode 2 – Design of concrete structures. Part 3: Liquid retaining structures and containment structures

Notes LCC = London County Council; ICE = Institution of Civil Engineers; RIBA = Royal Institute of British Architects

17.5.4 Concrete dams – historical development

The evolution of masonry/concrete dam design in the UK is summarised in Table 17.10.

Table 17.10 Evolution of masonry/concrete dam design in the UK

Data	Description
1871	Rankine's proposed no-tension rule for masonry (that is, classical middle third rule_
1882	First recognition of foundation uplift. Feature evolved from the Vyrnwy dam (1889) but was not universal – and then only on larger dams – until *c.*1955-1960.
1900	Mass concrete begins to replace traditional masonry construction, with masonry being used as facing.
1908	Following debate, classical design methods validated.
1909	Start of provision of transverse construction joints at 12–15m, but not universal, with waterstops, until late 1930s
1920s to 1950s	Massive buttress dam profiles developed

Notes: From Ledbetter et al. (1998), Hewlett et al. (2000), Chrimes (2009a, 2009b)

Existing design guides for gravity dams are listed in Table 17.11.

Relevant ICOLD Bulletins include Bulletins 71, 88, 107 and 145.

In the US, a database of concrete materials properties was examined to better estimate the average bond strength and the range of strength for dams constructed between 1905 and 1993 (Dolen 2011). The data analysis shows there is sufficient justification to apply separate strength input parameters in risk analysis for dams constructed with different state-of-the-art lift line preparation methods, corresponding to the periods prior to the early 1930s, 1930s to 1960s, and post 1970's onwards. Both the percentage of bonded lift lines and the average and range of strength are considered significant. It was for this reason, that although having good construction records including construction photographs, that Scottish and Southern decided to use risk assessment to assess the uncertainty in the tensile strength of lift joints (Mason 2010).

Table 17.11 Design guides for gravity dam properties and design

Date	Author	Title/comment
1921	A.H. Gibson	*Hydroelectric Engineering*, Blackie & Sons, London.
1958	J. Guthrie Brown (ed.)	*Hydro-electric Engineering Practice. Volume 1 Civil Engineering.* Blackie & Sons, London. Chapter VIIII covers gravity dams and Chapter XI covers buttress dams.
1976	Reclamation	*Design of Gravity Dams*, US Bureau of Reclamation, Denver, CO.
1996	Kennard et al.	*Engineering Guide to the Safety of Concrete and Masonry Dam Structures in the UK.* Report 148. CIRIA, London.
2009–2011	Reclamation	*Best Practices Training Manual.* US Bureau of Reclamation, Denver, CO.

17.5.5 Published papers on historical failure rates of concrete dams

The University of New South Wales research on dam safety included an analysis of statistics of failures and incidents (Douglas et al. 1998) as summarised in Table 17.12. It should be noted that the lack of failures (Note 1) means that many of the values are upper bounds and thus overestimates. Similarly the apparent difference between concrete and masonry may reflect different populations and is not necessarily a reflection of different risk.

The combination of lack of incidents and limited life years of experience with concrete dams in UK means it is not possible to carry out statistically significant assessment of UK incidents.

Table 17.12 Annualised failure rates for gravity dams

Failure Mode	Year Commissioned	Concrete Gravity		Masonry Gravity	
		0-5 years	>5 years	0-5 years	>5 years
All Failures	pre 1930	N/A	6.4E-05[2]	N/A	3.2E-04[2]
	1930-present	1.3E-04[2]	1.2E-05[2]	1.5E-03[2]	2.4E-04[2]
Foundation Sliding P_{SA}	pre 1930	N/A	5.0E-05[2]	N/A	6.0E-05[1]
	1930-present	2.0E-05[1]	4.0E-06[1]	5.0E-04[1]	2.0E-05[1]
Foundation Piping P_{PA}	pre 1930	N/A	7.0E-06[1]	N/A	6.0E-05[2]
	1930-present	2.0E-05[1]	4.0E-06[1]	5.0E-04[1]	2.0E-05[1]
Within Dam Body P_{BA}	pre 1930	N/A	7.0E-06[1]	N/A	2.0E-04[2]
	1930-present	9.0E-05[2]	4.0E-06[1]	5.0E-04[1]	2.0E-04[2]

Note: (1) No failures, probability estimated lower than that for one failure.
(2) Probability rounded down to account for the smaller than actual population used in the analysis.

Source: Douglas et al. (1998, Table 7.1)

ICOLD Bulletin 109 (ICOLD 1997, pp. 35-39) notes in relation to concrete dams:

a) 'The safety record of gravity dams built before 1930 was in fact worse than for embankment dams. The probability of failure was similar, but for gravity dams sudden failures caused more victims.

b) Overturning of blocks or sliding on the foundation (was) the most frequent cause.
c) Forty per cent of failures occurred on first filling.
d) Masonry dams built since 1930 have displayed similar safety performance as concrete dams.
e) Although arch dam experience trails gravity dams by 50 years, today's safety appears equivalent.
f) Buttress and multiple arch dams therefore appear to be less safe than gravity and arch dams.'

ICOLD Bulletin 88 (ICOLD 1993) indicates:

- Section 3.1.3 (page 117) presents annual probability of concrete dam failure (all causes, wear-in and in service) as 1.4×10^{-5} per dam year.

- Figure 31 (page 118) presents graphs of probability of failure against date of construction (all causes and foundation rock failure).

- Seventy-five per cent of concrete dam failures are due to foundation rock failure so overall probability of foundation failure 10^{-5} per dam year, of which only one third in service that is 3.3×10^{-6}.

Table 17.13 summarises published information on the relative annual probability of the different failure modes, which includes failure in the wear-in period and subsequently. It can be seen that overtopping, sliding and internal erosion of the foundation are broadly similar in annual probability.

In parallel with the work on embankment dams there was research on historical performance on concrete dams reported by Douglas et al. (1998, 1999). However, this had a relatively limited population of 487 incidents, comprising 46 failures, 176 accidents and 265 major repairs. The results of the statistical analysis are therefore considered less reliable than for embankment dams.

Published predictions of probability of failure of individual concrete/gravity dams are given in Table 17.14. The Tier 2 event tree method described in this Guide, when applied in a workshop process including ten service reservoirs and twelve concrete dams owned by one undertaker in UK, gave overall probability of failure of around an order less than the embankment dams owned by the same Undertaker, and was accepted as reasonable.

Table 17.13 Published information on relative annual probability of different failure modes for concrete dams

	ICOLD Bulletin 99, 1995 (Fig 10 & 12 in Bulletin) Note : below only includes primary cause of failure			Douglas, Spannagle & Fell, 1999 (Table 4)
Location, number of dams, and period of sample	World-wide, 17,400 number, all known failures prior to 1995			world-wide, 487 number
	No	Code	% of failures, exc. 'unknown'	
Mode of failure				
External erosion (Overtopping)	8	1.3.7, 3.4.6	33%	32%
Rupture of upstream dam	-	not	differentiated	
Internal erosion (foundation)	7	1.1.5, 3.1.5	29%	19%
Shear failure – foundation	9	1.1.3, 1.3.2, 3.1.3, 3.4.2	38%	22%
shear failure – dam body				19%
Appurtenant works	Excluded			
Other causes.				16%
Unknown cause	5			
	29			

Table 17.14 Published predictions of probability of failure of concrete/gravity dams

Dam	Reference	Predicted probability of failure	Comment
Loyne	Mason (2010)	3.8×10^{-7}	Combination of 1:10,000 year flood with event tree analysis of likelihood of failure on lift joint

18 External threats

This supporting information in respect of estimation of the probability of failure due to external threats comprises additional guidance and where appropriate summary of supporting science, subdivided into:

- methods of combining likelihood of external load and system response
- loads
- response of embankment dams to load
- response of concrete dams and service reservoirs to load

Where appropriate, a short description is included on the basis of the system incorporated in the guide.

18.1 Definitions

Following on from the definition of internal threats, ‘external threats’ are any action originating outside of the body of the dam, and thus include floods, earthquake, mining, subsidence and so on

18.2 Key issues and concepts

This section describes some of the key issues in relation to quantifying failure of dams due to external loads.

18.2.1 Probable maximum load

For high consequence dams, engineering standards sometimes require the design to remain elastic (or for earthquake to prevent release of the reservoir) at what is considered to be a physical maximum load at the given site. There is an increasing realisation that this may not be sufficient to guarantee safety. Reasons for this include:

- How probable is the probable in PMF? (Note there is now good evidence that probable maximum precipitation has been exceeded in over 10 events in the UK, particularly over small catchments.)
- Changes in climate may mean that historic assessment may not no longer be valid.

Thus risk assessment tends to assign an annual probability to what was historically considered a ‘probable maximum event’ with suggested values in Table 8.2.

18.2.2 Reservoir level vs. time

For many UK reservoirs it is reasonable to assume that the reservoirs is full for most of the year, so that the effect of lower reservoirs levels on reducing probability of failure can be neglected. This is certainty true of amenity lakes and also of some water supply reservoirs.

In arid parts of the world, where reservoirs are hydrologically larger (such that can store say twice mean annul run-off compared with UK reservoirs which are only sized for say a third mean annul run-off) are normally at half height and only fill every 20 years this would be invalid, and the analysis would need to be adjusted to take the reservoir stage duration curve into account.

18.2.3 Combining load and system response

This calculation is simplified to a single 'dam critical load' at Tier 1 and Tier 2, but for more detailed analysis distributions of load and response can be considered. This may provide a higher overall probability especially when the system response is non-linear with weaker point(s) where the probability of failure may be higher at lower magnitude more frequent loads.

A more detailed analysis can be done in several ways including:

- points either side of the dam critical load, giving the lower bound load condition for which failure will definitely not occur, and an upper bound condition for which failure will definitely have occurred
- manual subdivision of loading continuum into ranges of load each with their associated probability (the ANCOLD guidelines on risk assessment give a good example of this in Guidelines 6 and Appendix E; ANCOLD 2003)
- continuum of load and response, for example, as can be utilised through Monte Carlo analysis

18.2.4 Likelihood of failure: probability vs. safety factor

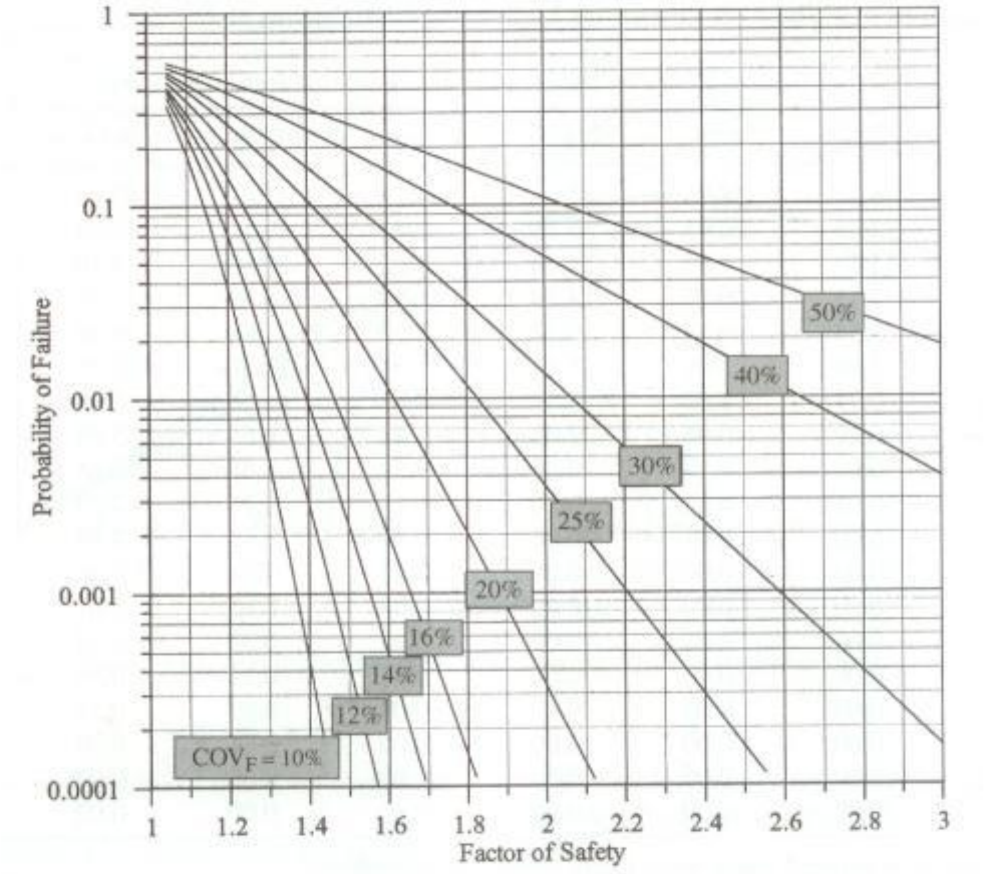
The likelihood of failure can be expressed in different ways – by a probability, or by a safety factor. At Tier 3, an assessment can be carried out using probabilistic-based methods, such as Monte Carlo analysis, to investigate the effect of both the median parameter and spread of parameters on probability of failure. It is important to realise that the probability of failure is significantly affected by the spread of data.

Published papers where authors have investigated this relationship include:

- Duncan and Wright (2005) – as illustrated in Figure 18.1, with the coefficient of variation (COV) for geotechnical parameters ranging typically between 5 and 40%
- Figure 30 in ICOLD Bulletin 88 (ICOLD 1993)
- FLOODsite EU research project (van Gelder 2008, Simm et al. 2008)
- Eddleston (2012)

However, at Tier 2 it is considered that direct estimation of probability of failure is too complex for this level of assessment, and instead this is simplified to safety factor, which is then converted to probability using Figure 8.4 (in Chapter 8).

Figure 18.1 Probability of failure based on lognormal distribution of safety factor



Source: Adapted from Duncan and Wright (2005, Figure 4.7)

18.3 Basis of tiered set of tools to estimate external loads – likelihood versus magnitudes

18.3.1 Introduction

Table 8.4 provides a summary of the methods of estimation of magnitude and annual chance of load for Tier 2, which is in general self-explanatory.

18.3.2 Basis of methodology for Tier 1

The methodology for Tier 1 has been derived by converting the Tier 2 methodology into a qualitative system, using the probability-descriptor relationship described in section 15.2.

18.4 Basis of tiered set of tools to estimate response of embankment dams to external load

18.4.1 Scour of slopes protected by vegetation

While there has been much research into the effects of vegetation and grass on flow within channels, the degree of guidance available on the performance of grass cover for levees or dams during overflow or wave overtopping conditions is far more limited. Guidance divides into grass performance under overflow conditions (often mis-quoted as overtopping) and performance under wave overtopping conditions. A review of current research and guidance for both can be found in the EU FloodProBE project report WP03-01-10-06 (Morris et al. 2013).

Research and guidance often originates back to three sources:

- research by US Department of Agriculture (USDA) at Stillwater, Oklahoma
- UK publications by CIRIA
- on-going research in the Netherlands into grass performance on levees during wave overtopping

There are notable differences in approaches from these sources. The US guidance looks at the combination of grass type and soil resistance to erosion while the UK guidance looks only at grass condition. The Dutch guidance focuses on wave overtopping, but applied to the performance of Dutch levees, which are normally constructed from a grass covered clay layer sitting over a sand core; performance analysis for the outer layer, however, should be generically applicable.

Grass performance under overflow conditions

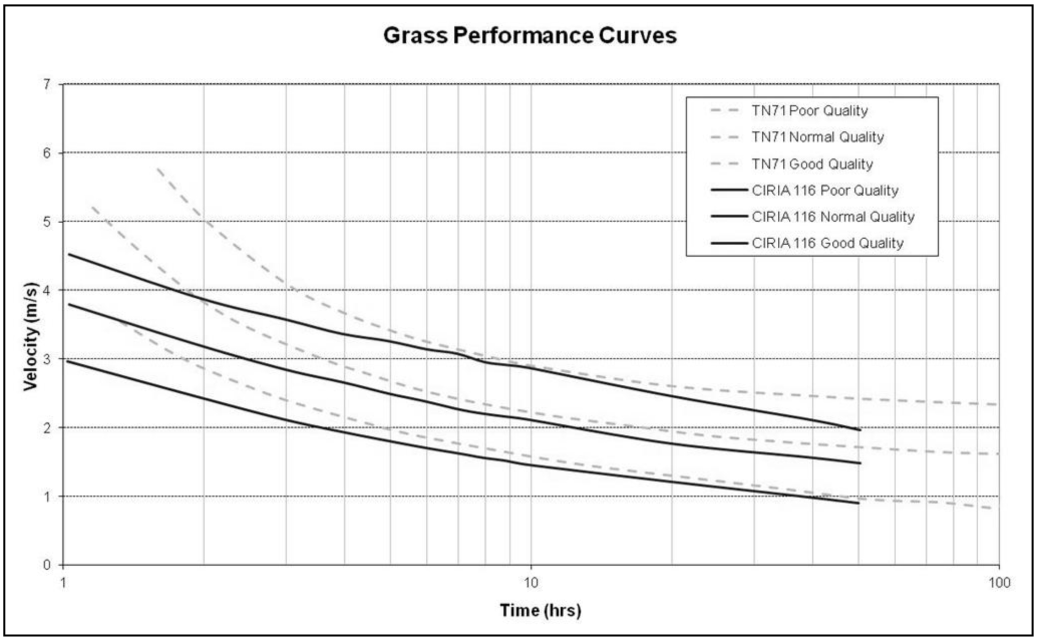
Existing guidance relates to two sources: European guidance often relates or refers to work by CIRIA during the 1970s and 1980s, drawing on CIRIA Technical Note 71 (Whitehead et al. 1976) or CIRIA Report 116 (Hewlett et al. 1987). Guidance in the US typically builds on the Agricultural Handbook 667 (Temple et al. 1987).

CIRIA guidance provides design curves which suggest acceptable limits for combinations of flow velocity and duration. The US approach estimates shear stress at the soil surface (as a function of vegetation type and impact) followed by acceptability in relation to the soil erodibility.

The design curves in CIRIA Report 116 appear to contain a factor of safety compared with the performance curves presented in the earlier CIRIA Technical Note 71 (TN71). Hence, while these may be appropriate for use in design, care should be taken to use the earlier TN71 curves when undertaking a performance assessment. If CIRIA Report 116 curves are used for a performance assessment, the results will predict poorer performance than may be reasonably expected as a result of the embedded factor of safety. Figure 18.2 shows a comparison of the TN71 and Report 116 design curves.

The approach recommended within this guide is the use of performance curves developed from the TN71 data. In particular, identification of the critical flow velocity on the embankment grass face that corresponds to the point when grass cover would fail. This condition is identified from the curves given in Figure 18.2 and then linked back to the reservoir level and hence flood event required to initiate such conditions.

Figure 18.2 Comparison between CIRIA 116 grass performance curves (Hewlett et al. 1987) and the original field test data (Whitehead et al. 1976)



Soil erosion leading to dam failure

Dam embankment erosion will typically be in the form of surface or headcut erosion, depending upon the nature of the soil. These processes are fuelled by the removal of sediment from the dam body and are discussed further below.

In this guide it has been assumed that if, the grass cover fails, soil will most likely erode to breach and failure, given sufficient time. The rate of soil erosion will depend on the soil type and state, but as indicative values, fairly compacted clay might start to erode with velocities in excess of 0.8m/s and a stiff clay in excess of 1.5m/s. These values are relatively low, for example, when looking at the data for grass performance shown above. Hence the assumption that erosion and failure would be likely to occur following grass failure is not unrealistic.

Soil erosion can occur via the three mechanisms listed below (de Vroeg et al. 2002, Mostafa 2003, Mostafa et al. 2008); this is also supported by observations from the IMPACT field test data (Morris 2009). These mechanisms comprise:

- sediment erosion
- mass erosion
- soil wasting

Sediment erosion occurs when sediment is removed from the surface of the embankment and held in suspension by the flow. Mass erosion occurs when small lumps of soil, rather than individual particles, are removed from the embankment surface by the flow. This process is particularly affected by the structure of the soil, including any fissuring that may have occurred. Soil wasting occurs when large blocks of soil are undercut and collapse into the breach flow. These are then quickly removed via a mixture of sediment and mass erosion (Figure 18.3).

Figure 18.3 Small scale erosion mechanisms

a. Sediment erosion by turbulent flow along base of breach sides

b. Mass erosion – small lumps of soil/clay being removed

c. Soil wasting – block failure on left face of breach

d. Soil wasting – block failure on left face of breach 2s after failure of block into breach (that is block has been removed)

These processes can be seen in different scales of dam or levee; for example, headcut and block failure during failure of the El Guapo dam shows similar processes to those seen during tests on 5-6m high levees (Figure 18.4).

Figure 18.4 Failure of the El Guapo Dam (Venezuela)

a. Headcut erosion back through dam after failure of the spillway (El Guapo Dam, Venezuela)

b. Vortices undercutting the breach sides just after failure of the spillway crest (El Guapo Dam, Venezuela)

The rate of dam erosion towards breach can be seen to be highly dependent upon soil state – for example, a highly compacted soil as compared to a loosely placed soil, will take much longer to erode. A common form of erosion equation that takes soil erodibility into consideration is given below:

$$\boldsymbol{E = K_d b(\tau - \tau_c)^a} \qquad \text{(Eq. 18.1)}$$

where:

E = erosion rate, bulk volume hence rate of bed elevation change or retreat ($m^3/s/m^2$)

K_d = erodibility or detachment coefficient (-)

τ = effective shear stress (kPa)

τ_c = critical shear stress (kPa)

a, b = empirical coefficients dependent upon soil properties (-)

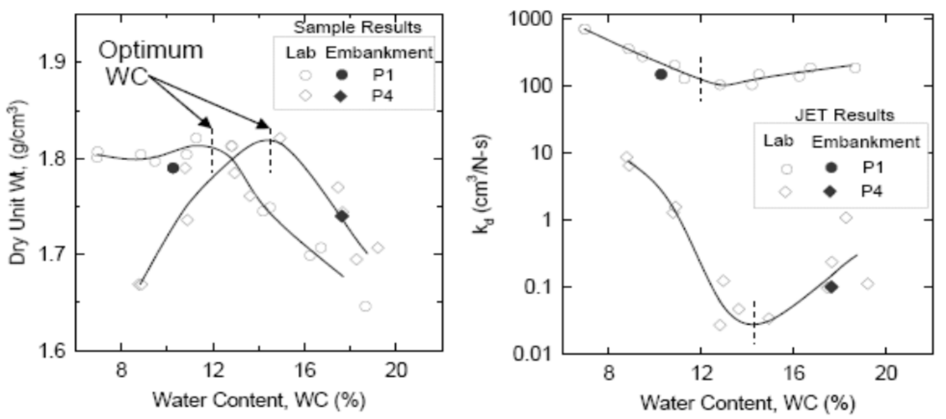
Assumed that a = b = 1 (Hanson et al. 2005), the only variables in calculating the rate of erosion are then the critical shear stress (τ_c) and the erodibility of the soil (Kd). The use of such an erosion equation has two advantages. First, the equation reflects a dynamic erosion process and is not based upon steady state equilibrium conditions which clearly do not apply. Secondly, the erodibility parameter, Kd, can be used to reflect variations in erosion as a function of soil state (compaction, moisture content and so on). It can be seen that soil erodibility is highly dependent upon soil compaction and moisture content (Figure 18.5).

The drawback to using an equation based upon an erodibility coefficient, such as Kd is the need to define a value for Kd. To date this has been undertaken through laboratory or field testing (Hanson and Cook 2004). The two main approaches are jet testing (JET – Hanson) for erodibility relating to surface or headcut erosion and hole erosion testing (HET – Fell) for internal erosion erodibility.

Figure 18.5 Example analyses showing relationship between soil erodibility (K_d) and soil type, density and water content

Embankment Test	Soil Classification[1]	Sand[1] > 75μm (%)	Fines[1] > 2μm (%)	Fines[1] < 2μm (%)	PI[2]
P1	SM	64	29	7	NP
P4	CL	25	49	26	17

Note: Based on ASTM : [1]D 2487, [2]D 4318.



Source: Hanson et al. (2010)

Simple guidance on the likely range of erodibility for a given soil and state is available, but this is indicative and care should be taken to assess the impact of uncertainty in these values on any particular study. Temple and Hanson have undertaken programmes of research into soil and vegetation performance at the USDA Agricultural Research Service Centre at Stillwater, Oklahoma. As part of this work they have produced some indicative and qualitative descriptions of soil erodibility, as shown in Equation 18.2, Table 18.1 and Table 18.2. Equations 18.2 to 18.4 provide an approximate method for estimating erodibility (Kd) based upon percentage clay content and soil density (Temple and Hanson1994).

$$\boldsymbol{K_d} = \frac{10\gamma_w}{\gamma_d}\exp\left[-0.121(\boldsymbol{C}\%)^{0.406}\left(\frac{\gamma_d}{\gamma_w}\right)^{3.10}\right] \qquad \text{(Eq. 18.2)}$$

where:

K_d = erosion rate in units of [(cm^3/N-s)]

$C_{\%}$ = percentage clay

γ_d = dry unit weight in mg/m^3

γ_w = unit weight of water in mg/m^3

Equation 18.3 (Hanson et al. 2007) provides an (unpublished) indicative equation relating K_d to compaction energy and moisture content of the soil:

$$\boldsymbol{k_d} = 8.11x10^9\,\boldsymbol{E_c}^{-0.5}\boldsymbol{WC}\%^{-7.0} \qquad \text{(Eq. 18.3)}$$

where:

K_d = erosion rate [ft/hr/(lb/ft^2)]

E_c = compaction effort (ft-lb/ft^3)

WC% = compaction water content per cent

When using Equation 18.1 a value of τ_c is also required. An approximation is to assume that $\tau_c = 0$ or to use Equation 18.4 (Hanson and Simon 2001, Hanson and Hunt 2007).

$$\boldsymbol{K_d} = 0.2\boldsymbol{\tau_c}^{-0.5} \qquad \text{(Eq. 18.4)}$$

where:

K_d = erosion rate (cm^3/N-s)

τ_c = critical shear strength (Pa)

Given the uncertainty associated with a clear description and measure of erodibility, an alternative approach is to adopt qualitative descriptions of erodibility and to allow for this uncertainty when considering modelling results. Tables 18.1 and 18.2 provide examples of such qualitative descriptions.

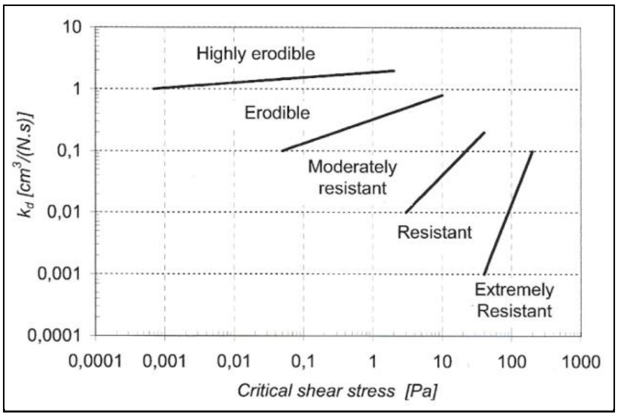
Table 18.1 Qualitative descriptions of values for K_d

Qualitative description of values for K_d (modified from Hanson, et al. 2007)		
K_d (ft/h)/(lb/ft^2)	**Description**	**K_d** (cm^3/N-s)
>10	Extremely erodible	>18
1–10	Very erodible	1.8–18
0.1–1	Moderately erodible	0.18–1.8
0.01–0.1	Moderately resistant	0.018–0.18
0.001–0.001	Very resistant	0.0018–0.0018
<0.001	Extremely resistant	<0.0018

Table 18.2 Factors affecting soil erodibility *(*Hanson 2007)

% Clay	**Well compacted (ft/h)/(lb/ft^2)**		**Poorly compacted (ft/h)/(lb/ft^2)**	
	At optimum moisture content	**Dry of optimum moisture content**	**At optimum moisture content**	**Dry of optimum moisture content**
	K_d	K_d	K_d	K_d
>25	0.1	1	1	10
10–25	0.5	5	5	20
5–10	2	10	10	50
0–5	10	20	20	100

Figure 18.6 Erodibility of soil



Source: Hanson and Simon (2001)

18.4.2 Hydraulics of flows in spillway chutes

The hydraulics of supercritical flow in spillway channel is a complex subject, especially where there are changes in vertical and horizontal alignment, and steps in the invert. A simplified method has been devised to estimate when there is sufficient out of channel flow to erode the adjacent embankment, as described in the guide for Tier 2 analysis. Some of the key simplifications are summarised in Table 18.3.

Table 18.3 Key assumptions and simplifications in in Tier 2 methodology for spillway chute capacity

	Issue	Assumption	Implication/comment
Base methodology included in Tier 2			
1	Velocity sufficient cause scour of embankment adjacent to chute	Assume governed solely by quality of grass and slope of dam.	Slightly unsafe – in reality there will be a shear zone along the top of the wall, between high velocity flow in the chute and slower flow on the bank.
2	Velocity in chute	Normal flow is governed by slope at that point, not depth below weir.	Flow conditions will be complex, being affected by both local slope and geometry/ flow conditions upstream of the point in question.
3	Effect of steps	Neglect (that is, consider base of chute as being of nose on steps)	Reasonable for 'dam critical flow' that is flow regime on steps likely to be skimming and not nappe flow.
4	Correction for whitewater	Assume bulking factor of 20%	In reality will vary with steps, bends, velocity and material forming sides of channel. This is selected as a 'typical value'. User is free to select a different value.
5	Direct impact of jet onto embankment face	Neglect at Tier 2. If significant issue then move to Tier 3.	
Optional User extension to deal with bends			
6	Bends – how is radius determined?	Bend radius R is based on bend angle and chute width, defining the radius as starting at points 2W upstream and downstream of intersection of the centreline of the chute, where W is average chute width around bend.	Geometry of bends is in reality quite complex and can include contraction/expansion of the channel width as changes in bed slope. This is a simplification to allow hand/Excel calculation for screening purposes, rather than full computational fluid dynamics (CFD) model.
7	Bends – formula for super elevation	Use ASCE formula: $y = 0.35\ V^2\ W/\ gR$, where V is velocity, W is channel width and R is effective channel radius.	There are various published formulae. In reality the super elevation will depend on the angle and detailed geometry of the bend in relation to channel profile, such that any formula can only be an approximation, with detailed model testing (physical or CFD) necessary for a better estimate.

18.4.3 Stability of soil slopes (embankment)

The level of complexity of calculation which is appropriate for slope stability calculations escalates broadly as follows:

- precedent/ local knowledge
- hand wedge calculations
- stability charts (see for example Spencer 1967, Bishop and Morgenstern 1960)
- limit equilibrium
- finite element stress analysis
- dynamic analysis

Stability analysis normally considers conservative parameters to reflect the weakest point in the dam or uses Monte Carlo analysis with a credible range of parameters. The level of detail of data on the structures required to support this analysis similarly escalates, and includes (where available):

- specification for original construction
- construction records, including photographs
- intrusive investigation and associated laboratory testing

For many of the reservoirs there is no stability analysis available, so the simplified approach is as shown in Table 18.4

Table 18.4 Tiered approach to probability of release of reservoir due to embankment slope instability

Feature	Tier 1	Tier 2
Probability of applied load	Use Table 4.4	Use Table 8.4
Probability of slope failure for given load	Relate to defined slopes for 'modern slope design' Table 4.5	Stability index charts (Figures 8.9 and 8.10), based on assumptions in Table 18.5
Conditional probability of failure of reservoir, given slope instability	Use Table 4.5	See Table 8.7

Table 18.5 Key assumptions made in Tier 2 embankment stability charts

Feature	Issues	Simplified approach adopted to derive stability index
Mode of failure/ method of analysis	Depends on whether failure in body of embankment, or in foundation	Hand wedge calculations assuming thrust of $0.7\gamma H^2/2$
Geometry		
Dam		Overall slope angle of downstream face
Crest	Wider crest reduces risk of failure surface that would extend into reservoir	Neglect
Freeboard	Affects position of phreatic surface (along with depth to top of watertight element)	Neglect
Material properties		
Bulk density	Fill, foundation	Assume $18k/m^3$
Shear strength	Drained or undrained analysis, properties	Assume effective cohesion is zero, shear strength as shown on graph
Foundation	Assume no weaker than fill	
Loads		
Upstream water; internal pore pressure distribution		Represent by Ru (ratio of average pore pressure to total stress)
Tailwater		Not included
Live load on dam crest		Neglect
Earthquake		As Table 8.4

Table 18.6 Basis of guidance on assessing Tier 2 conditional probability of failure

Feature		Basis/ comment
Base conditional probability	1 in 3000	**(a)** Ratio of number of slope incidents to slope incidents leading to reservoir release **(b)** Alternative approach would be to carry out a vent tree analysis for a number of dams, and use average value from that process **(c)** This value is a judgment by authors of Guide, as somewhere between 1 in 1000 and 1 in 10,000
Adjustment factors	Vary	Some of these could be obtained by FE analysis, looking at the magnitude of ground movement needed for the dam crest to be displaced to below the reservoir level. However, for Tier 1 and Tier 2 analysis, applicable to a wide range of dams, typical numeric values based on the experience of the Panel AR authors of the guide are adopted.

18.5 Basis of tiered set of tools to estimate response of concrete gravity structures to external load

18.5.1 Stability of concrete gravity structures

Methods of calculation to assess the stability of gravity sections are given in references such as:

- *Design of Gravity Dams* (Reclamation 1976)
- US Association of Dam Safety Officials 'Training Aids 'Review 4 Evaluation of Concrete Dam Stability' (ASDSO 1988)
- CIRIA Report 148 (Kennard et al. 1996b).

The level of complexity of calculation which is appropriate escalates broadly as follows

- simple stress distribution for triangular section from water line
- stability charts (none published for concrete structures, because of the large number of variables)
- rigid block stability calculations
- finite element stress analysis
- dynamic analysis

Stability analysis normally considers conservative parameters to reflect the weakest point in the dam, or use Monte Carlo analysis with a credible range of parameters. The level of detail of data on the structures required to support this analysis similarly escalates, and includes (where available):

- specification for original construction
- construction records, including photographs

- intrusive investigation and associated laboratory testing

For many of the reservoirs there is no stability analysis available, so the simplified approach shown in Table 18.7 has been developed.

Table 18.7 Tiered approach to probability of release of reservoir due to instability of gravity structure

Feature	Tier 1	Tier 2
Probability of applied load	Floods : L1 is 1 in 10, L2 is 1 in 100, and so on Earthquake – consider 0.28g and assume 1 in 30,000 chance	Use Table 8.4
Probability of stability failure for given load	Floods Simplified from Tier 2 to four ranges of likelihood, as shown in table 4.3.4, based on annual chance of 1 in 2, 20 and 200 Effect of earthfill providing support – Judgement, based on experience of application of event tree approach.	Stability index charts (Figures 8.12 to 8.16), based on assumptions in Table 18.8
Conditional probability of failure of reservoir, given stability of gravity wall	1 in 10	Provided by phases in event tree

Table 18.8 Key assumptions made in Tier 2 gravity structure stability index charts provided in Part 1

Feature	Issues	Simplified approach adopted to derive stability index
Mode of failure/ method of analysis	Sliding Overturning/ bearing pressure Stress analysis (tensile stress on face, position of resultant)	For simplicity consider rigid block sliding only, using conservative parameters (Note 1)
Geometry		
Dam		A – width at upstream water level B – width at foundation C – freeboard
Embedment below ordinary ground level (OGL)	Ground level Lateral stresses, including relationship to lateral deformation of dam	Neglect (that is, consider lift joint)
Landscaping fill on downstream face	Present, and if so what pressure, for example, Ko (but could increase to Kp with large deformation)	Neglect

Feature	Issues	Simplified approach adopted to derive stability index
Material properties		
Bulk density	Mass concrete, masonry	Assume 23kN/m^3
Concrete	Strength properties Elastic properties	Assume High that is > compressive stresses
Lift joints	Tensile strength Shear strength Pre-existing cracks – length/ aperture	Neglect Assume ϕ' = 45 degrees c′ = 0 Neglect
Foundation (rock)	Tensile strength Shear strength	Adjust parameters for index stability to reflect likely foundation strength
Loads		
Upstream water	Reservoir height above founding level (For Service reservoirs limited to height above lined floor)	H – variable on Figures 8.12 to 8.16
Tailwater	Stream/ groundwater	Neglect
Internal pore pressure distribution	Dependent on • presence/ position/ effectiveness of internal drains • Pre-existing cracks which could have full uplift	Triangular. Neglect refinements listed
Deadweight above dam faces	Water if upstream face inclined into reservoir	Assume upstream face vertical
Wind and wave		Neglect
Ice loads		Neglect
Earthquake		As Table 8.4

Notes: [1] Critical failure modes implied by calculations likely to vary depending on parameters selected for example higher shear strength means that cracking/ tensile strength become the critical case. So although the stability ranking index reflects reflect sliding on a poor quality lift joint, it is also intended to be an overall assessment of stability, not just sliding.

18.6 Supporting information

18.6.1 Wave forces (Tier 3)

Introduction

It has been appreciated for many years that apparently similar wave conditions may give rise to dramatically different wave pressures or forces depending on the form of wave breaking at, onto, or close to a vertical wall. Under wind waves, there will inevitable be a wide range of wave breaking, but it is generally convenient to use three or four categories of wave load/breaking condition:

- non-breaking or pulsating
- impulsive breaking or impact
- broken waves
- post-breaking or bore waves

The simplest case, is generally when the wave is non-breaking, also termed reflecting or pulsating. For this condition, the wave motion is relatively smooth and the main processes can be predicted by simple wave theories.

Much more intense wave forces/pressures arise if the wave can break directly against the wall – termed plunging, breaking, impulsive or impact.

It is recommended that forces that act on floodwalls be calculated for the design water level as well as for water levels equal to the top of the wall crest and at the maximum possible water level that results in overflow, if applicable. The critical loading case to be considered for design should be where h equals the full height of the wall or the highest anticipated water level if greater than the wall height.

Impulsive breaking is strongly influenced by any mound, berm or steep bed slope in front of the wall; conditions are difficult to predict and attract significant variability/uncertainty. In the past, these variations have led to significant lack of clarity in advice on wave forces.

Rather lower forces arise if waves have already broken before reaching the wall. The wave motion is turbulent, but often highly aerated. Predictions of broken wave loads are uncertain, with relatively few laboratory or field data.

The last type is the post-breaking or bore wave, which usually applies to a wall whose toe is above the static water level, but where the run-up bore can still reach the wall.

Broken waves occur when the local water depth is insufficient to support unbroken waves. For simple vertical walls with no significant mound, waves may start to break when the local wave height to depth exceeds (say) I/d >0.35. As local wave conditions approach the breaking limit, so the proportion of broken waves increases, and the probability of a large (but un-broken wave) reduces.

Sources of methods for the calculation of wave forces

Methods for calculating wave forces on vertical walls can be categorised into:

- pulsating (or non-impulsive) wave loads;
- impulsive breaking or impact;
- broken waves
- post-breaking or bore waves
- pulsating (or non-impulsive) wave loads

The main default method to calculate quasi-static wave loads should be Goda's, or Takahashi's modified version of Goda's method. The most robust (and most widely accepted) prediction method for wave loads on vertical and composite walls is that developed by Goda (1974, 1985). This method assumes that wave pressures on the front face can be represented by a trapezoidal distribution, reducing from p1 at the static water level (SWL) to p3 at the wall base.

The simple prediction methods for pulsating wave loads by Goda or Ito (Ito 1971) generally predict average pressures up to about pav = 2ρgHs where Hs is the incident (local) significant wave height.

Impulsive wave loads

A simple and robust method to predict wave impact pressures was derived by Allsop and Vicinanza (1996) based on testing by Allsop et al. (1996a). They noted that for waves close to breaking given by 0.35 < Hsi/d < 0.6; other prediction methods under-estimate measured forces.

Research studies in Europe have measured local wave impact pressures up to or greater than pimpact = 40ρgHs, much higher than would be predicted by simple design methods – see especially Allsop and Vicinanza (1996) and Allsop et al. (1996b). In extremis, tests by Kirkgoz (1995) suggest impact pressures up to impact = 100ρgH, although these are highly unlikely in practice.

Broken wave conditions

A method to estimate an average wave pressure from broken wave loads was developed by Blackmore and Hewson (1984). For some reservoirs, the design wave condition may be limited by depth in front of the structure. In these cases, the larger waves at the structure will be broken and it is most unlikely that wave impact loads will be caused.

Bore wave conditions

Where the (toe of the) wall is above the static water level, there is a single method cited in the Coastal Engineering Manual (USACE 2006) developed by Camfield (1991) based on earlier work by Cross (1967) for wave loads on back-beach seawalls. The method requires a wave run-up limit on the beach slope to be calculated, from which a wave ‘surge height’ (Hw) at the wall is deduced. Wave run-up levels are subject to significant measurement uncertainties and therefore to some debate. The classic method for estimating wave run-up on shallow slopes or beaches is that ascribed to Hunt (1959), perhaps as re-stated by Battjes (1974).

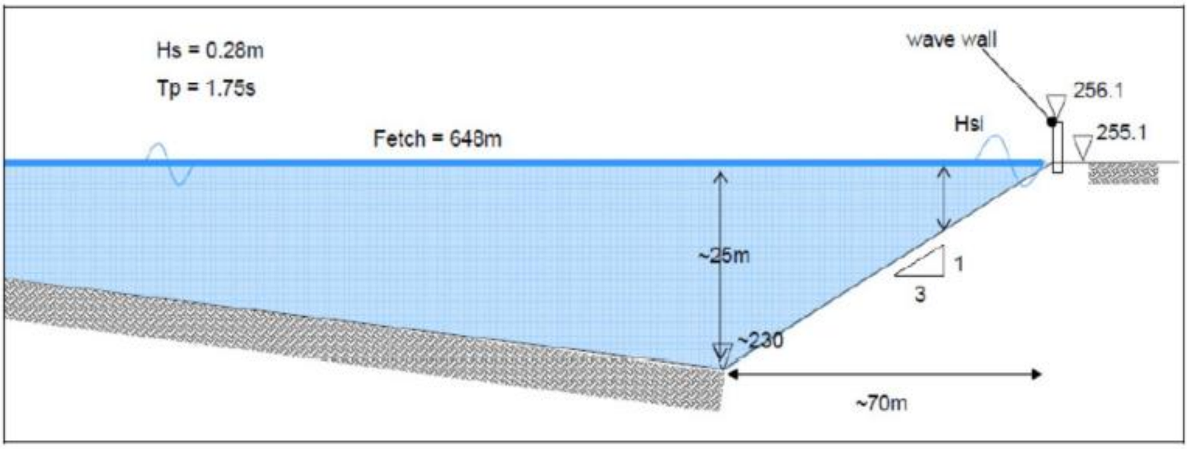
This method gives no indication of the height over which the load applies, nor of the average pressure, and so a simple rectangular distribution over the full wall height is generally assumed. The calculation of bore wave load is therefore rather subjective and it is not known whether it has been validated by any measurements, either field or laboratory. Its reliability is therefore unknown.

Case study of wave forces on a reservoir wall

An embankment dam at the western end of a reservoir faces approximately east to south-east. Prevailing winds are generally away from the dam, but waves along the main fetch (650m) of the reservoir may break directly onto the 1m high vertical wave wall at the crest of the 1:3 embankment slope.

There is no simple prediction method for wave forces on this wall (that is, within range for the particular geometry of dam slope, wave wall position and water level). None of the usual prediction methods are strictly valid for the particular configuration given. A number of methods therefore were used, all involving extrapolation from their original ranges. The key ‘given’ data are summarised in Figure 18.7.

Figure 18.7 Schematic of dam and wave wall – input conditions for the calculations



The water level coincides with the toe of the 1.0m high vertical wall and the crest of the embankment slope. This coincidence is unfortunate as no generic prediction method for either vertical walls or plane slopes is within range, hence the need to extrapolate different methods out of their intended range.

The adopted approach to calculating the wave load is as follows.

- Determine the effective wave condition at $5H_s$ seaward from the structure.
- Calculate the momentum-driven horizontal ‘Goda’ load (F_{hGoda}) and pressures.
- If the geometry has a noticeable berm, which may cause impulsive breaking, then apply the Takahashi modification to Goda’s method to give an enhanced quasi-static load of $F_{hG\&T.}$
- If impulsive wave loads are possible, then use simple methods by Allsop and Vicinanza (1996), or Cuomo et al. (2010a, and 2010b), to estimate $F_{Impulsive}$ and an impulsive load duration.
- If the wave can be broken by the time it reaches the wall, use the method of Blackmore and Hewson to calculate $F_{hB\&H.}$
- If the wall is only reached after a breaking/broken bore has travelled over a slope or beach, estimate the load by Camfield’s method, $Fh_{Camfield.}$

The default load should always be F_{hGoda} or $F_{hG\&T}$, either of which may be taken as a quasi-static load. Any impulsive load should be taken as an additional load case, not replacing the default load. High-intensity impulsive loads are limited in duration so must be treated as dynamic loads.

Assumptions and results

In the first stage, a check is made on wave conditions at positions from the dam toe to a depth of 0.1m below the wall toe. (NB: Extending the calculations to the wall would simply give zero wave height in zero water depth, a pointless calculation.) The ‘Goda’ location of $5H_s$ away from the wall toe was position 8 in these calculations with a ‘bed’ level at 254.6mODN.

There are no validated methods to predict shoaling and depth-limited breaking on a 1:3 slope, so calculations of incident wave height in Table 18.9 used a simple depth-limiting check for the steepest slope available at 1:10 to test whether waves will have broken before or at the analysis position.

Table 18.9 Summary wave condition check

Position	Bed level (mODN)	Local depth (m)	H_{si} (m)	H_{max}(m)
6	254.2	0.9	0.28	0.50
7	254.4	0.7	0.28	0.50
8	**254.6**	**0.5**	**0.28**	**0.50**
9	254.8	0.3	0.24	0.44
10	255.0	0.1	0.12	0.22

Wave conditions in Table 18.9 were then used to calculate Goda momentum-driven wave loads given in Table 18.10. These calculations inherently assume that the wall is shifted 'waterwards' such that the wall toe is below water level. The wall height used to calculate the total horizontal force will therefore be over-estimated, as will the calculated values of $F_{h1/250}$ itself. The indicative wave pressure at the waterline, p_1, will not however be significantly distorted by these (slight) changes to the structure geometry.

Table 18.10 Goda wave load check

Position	Bed level (mODN)	Local depth (m)	H_{max} (m)	$F_{h1/250}$ (kN/m)	P_1 (at SWL) (kN/m^2)
6	254.2	0.9	0.50	3.6	3.5
7	254.4	0.7	0.50	3.6	3.9
8	**254.6**	**0.5**	**0.50**	**3.7**	**4.7**
9	254.8	0.3	0.44	3.4	5.6

It is interesting to note that, while values of the wave pressure at the water line may increase 'landward' of position 8 (see, for example, p_1=5.6kN/m2 at position 9), this does not increase the total horizontal force, improving confidence in the calculation of $F_{h1/250}$ = 3.7 kN/m as the representative quasi-static loading at position 8. As impulsive breaking is likely, the Takahashi extension of Goda's method was applied for an (assumed) berm of 0.2m height and 0.25m width. The changes to $F_{h1/250}$ and p_1, however, are small (Table 18.11).

Table 18.11 Goda andTakahashi wave load check

Position	Bed level (mODN)	Local depth (m)	H_{max} (m)	$F_{h1/250}$ (kN/m)	P_1 (at SWL) (kN/m^2)
6	254.2	0.9	0.50	3.80	3.36
7	254.4	0.7	0.50	3.78	3.64
8	**254.6**	**0.5**	**0.50**	**3.81**	**4.11**
9	254.8	0.3	0.44	3.12	4.15

In the last set of calculations summarised in Table 18.12, methods by Allsop and Vicinanza for impulsive loadings, Blackmore and Hewson for broken waves, and Camfield for wave bores were applied. The calculation of broken wave loads with Blackmore and Hewson used a coefficient λ= 0.5, and the bore wave load calculated by Camfield used a Hunt wave run-up limit for H_s. As expected, the impulsive loads (Allsop and Vicinanza) increase as the depth decreases, while the broken wave load (Blackmore and Hewson) reduces with reducing depth. Load estimations using Camfield's method are very much lower than Goda's loads, and are not regarded as realistic.

Table 18.12 Impulsive, broken waves, and wave bore load check

Position	Bed level (mODN)	Local depth (m)	Allsop and Vicinanza		Blackmore and Hewson		Camfield
			$F_{A\&V}$	P_{av}	$F_{B\&H}$	P_{av}	$F_{Camfield}$
			(kN/m)	(kN/m^2)	(kN/m)	(kN/m^2)	(kN/m)
6	254.2	0.9	6.9	4.2	13	7.7	0.45
7	254.4	0.7	8.5	5.8	8.7	6.0	0.45
8	**254.6**	**0.5**	**11.1**	**8.8**	**5.4**	**4.3**	**0.45**
9	254.8	0.3	11.5	12	2.5	2.6	0.34

Recommendations

Given the unusual configuration (for wave load calculations) and the potential for plunging wave action onto the wall, the minimum load that should be considered should be the Goda load of $F_{h1/250}$ = 3.7 kN/m, taken as a quasi-static load. The possibility of two alternative loads (associated with broken waves, or plunging or impulsive waves) should, however, also be considered. If it can be demonstrated that these waves will break before the wall, then the broken wave load of $F_{B\&H}$ = 6.4 kN/m should be applied, taken as effectively a static load.

If however the wave can plunge direct against the wall, then impulsive loads should be estimated, for example, $F_{A\&V} \approx 11$ kN/m, $p_{av} \approx 9$kN/m^2. This will, however, only be of short duration, so must not be applied as a static load, but as an impulsive load with appropriate duration.

Sources of further guidance and science

- Allsop, N.W.H., 2000. Wave forces on vertical and composite walls', in *Handbook of Coastal Engineering*, ed. J. Herbich, pp. 4.1–4.47. McGraw-Hill, New York.
- Allsop, N.W.H. and Vicinanza, D., 1996b. *Wave impact loadings on vertical breakwaters: development of new prediction formulae*. In Proceedings 11th International Harbour Congress, Antwerp, Belgium.
- Allsop, N.W.H., Kortenhaus, A., McConnell, K.J. and Oumeraci, H., 1999. *New design methods for wave loadings on vertical breakwaters under pulsating and impact conditions*. In Proceedings Coastal Structures'99, Santander, Spain. Balkema, Rotterdam.
- Allsop, N.W.H., McKenna, J.E., Vicinanza, D. and Whittaker, T.J.T., 1996a *New design formulae for wave loadings on vertical breakwaters and seawalls*. 25th International Conference on Coastal Engineering, September 1996, Orlando, FL. ASCE, New York.
- Allsop, N.W.H., Vicinanza, D. and McKenna, J.E., 1996b. *Wave Forces on Vertical and Composite Breakwaters*. Research Report SR 443. HR Wallingford, Wallingford.
- Battjes, J.A., 1974. *Surf similarity*, Proceedings 14th Coastal Engineering Conference, Copenhagen, pp.466–479. ASCE, New York.
- Blackmore, P.A. and Hewson, P., 1984. Experiments on full scale impact pressures. *Coastal Engineering*, 8, 331-346.

- Camfield, F.E., 1991. Wave forces on wall, *Journal of Waterway, Port, Coastal, & Ocean Engineering*, 117(1), 76-79.
- Cross, R.H., 1967. Tsunami Surge Forces, *Journal of the Waterways & Harbors Division, Proc. ASCE*, 93 (WW4), 201-231.
- Cuomo, G., Allsop, N.W.H. and Takahashi, S., 2010a. Scaling wave impact pressures on vertical walls. *Coastal Engineering*, 57(6), 604-609.
- Cuomo, G., Allsop, N.W.H., Bruce, T. and Pearson, J., 2010b. Breaking wave loads at vertical sea walls & breakwaters. *Coastal Engineering*, 57(4), 424-439.
- Cuomo, G., Piscopia, R. and Allsop, N.W.H., 2011. Evaluation of wave impact loads on caisson breakwaters based on joint probability of impact maxima and rise times. *Coastal Engineering*, 58(1), 9-27.
- Goda, Y., 1974. *New wave pressure formulae for composite breakwaters*. In: Proceedings14th International Conference on Coastal Engineering, pp. 1702–1720. ASCE, New York.
- Goda, Y., 1985. *Random Seas and Maritime Structures.* University of Tokyo Press, Tokyo.
- Hunt, J.A., 1959. Design of seawall and breakwaters. *Journal of Waterway, Port, Coastal, & Ocean Engineering*, 85, 123-152.
- Ito, Y., 1971. Stability of mixed type breakwater – a method of probable sliding distance. *Coastal Engineering in Japan*, 14, 53-61.
- Kirkgoz, M.S., 1995. Breaking wave impact on vertical and sloping coastal structures. *Ocean Engineering*, 22(1), 35-48.
- Takahashi, S. Tanimoto, K. and Shimosako, K., 1994. *A proposal of impulsive pressure coefficient for the design of composite breakwaters*. In: Proceedings Hydro-Port ‘94, Yokosuka, Japan.
- USACE, 1989. *Engineering and Design of Retaining and Flood Walls.* Engineer Manual 1110-2-2502. US Army Corps of Engineers, Washington DC.
- *Coastal Engineering Manual*. Engineer Manual 1110-2-1100. US Army Corps of Engineers, Washington DC.

18.6.2 Ice loads

CIRIA Report 148 (Kennard et al. 1996b) quotes a value of 150kN/m run for the expansion of an ice sheet 600mm thick, but suggests that such thicknesses are unlikely to be encountered in the UK and a lower force for a 400mm thick sheet is probably the maximum that can be envisaged for severe conditions. For screening level purposes, and pending further research, ice loads may be assigned the following likelihoods shown in Table 18.13. Table 8.14 gives the suggested likelihood of ice loading adopted for the screening studies.

Table 18.13 Factors determining magnitude of ice forces on concrete dams

Factor	Comment
Ice thickness	The growth rate for ice cover can be estimated by the so called degree day method (ICOLD Bulletin 105 Equation 2.1; ICOLD 1996). This suggests that, for a 400mm thick ice sheet to develop, it requires ~180 degree days of freezing, equivalent to an average air temperature of -5 degrees for 36 days, or -10 degrees for 18 days.
The amount and the rate of change of temperature increase; Heat transfer at the top surface and in the ice sheet	Although methods are given in USACE 2002 for these factors, the ICOLD Bulletin provide a simplified equation for ice load related to ice thickness – see below.
Stiffness of boundaries resisting expansion of an ice cover	Equations in the Ice Manual suggest a 5° temperature rise (for example, from -5 to zero) in a 150m long ice sheet would result in a ~40mm increase in length. This is greater than the likely elastic response of a concrete dam on rock, such that load is likely to build up when rapid thawing occurs to thick ice sheets
Ice load	ICOLD Bulletin 105 Equation 3.8 provides a basis for estimating ice loads when ice cover is not perfectly restrained with: p = 245.c.h^0.5. where c is a coefficient assumed equal to 0.5 (as given in ICOLD Bulletin 105), h = ice cover thickness (m) and p = max thermal thrust per unit length (kN/m).
Likelihood of repeated thawing/freezing leading to cumulative displacement	Although ice loading may cause cracking partway through the structure and possibly displacement of the structure, the act of displacement/cracking is likely to reduce the load, such that failure may not occur on the first occasion but only after several applications of ice load

Table 18.14 Suggested likelihood of ice loading adopted for screening studies

Loading	Ice thickness	Degree days freezing needed	Likelihood of initiation
100kN	400mm	180	0.1% – Virtually impossible
50kN	200mm	44	1% – High unlikely

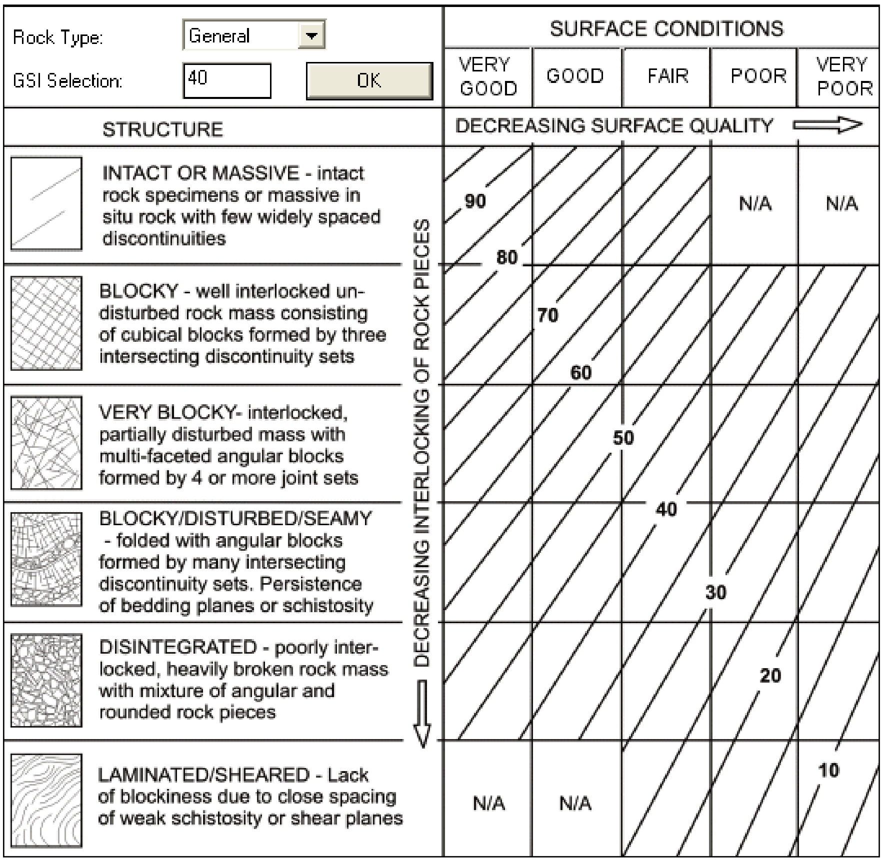
18.6.3 Rock mass shear strength

The foundation shear strength will depend on the properties of the rock mass and is stress dependent, reducing at higher confining stresses. A method of estimation is provided in Hoek et al. (2002), with a credible range of parameters being summarised in Table 18.15.

Table 18.15 Credible range of rock mass parameters for gravity dams founded on rock

Case			Poor (5% of cases?)		Good (95%?)	
Name	**Symbol**	**Units**	**Value**	**Basis**	**Value**	**Basis**
Input parameters						
Unconfined compressive strength	UCS	MPa	20	Weak, equivalent to mass concrete	60	Stronger
Geological strength Index (Figure 18.8)	GSI		20	Poor, blocky/ disturbed	40	Fair, Very blocky
Material constant	Mi		7	Siltstone		As 5%
Disturbance index	Di		0.7	Machine excavation	0.5	As 5%
Upper limit of confining (lateral) stress	σ'_{3max}	kPa	200	10m high dam		As 5%
Shear strength parameters (from Roclab, using Hoek et al. 2002))						
Angle of internal friction	ϕ'	degrees	28		47	
Effective cohesion	c'	kPa	30		130	
Shear stress at normal stress of 200kPa		kPa	140	Equivalent to instantaneous phi' of 35 degrees	340	

Figure 18.8 Geological strength index (refined version of rock mass rating) for rock mass



19 Dam break and flood routing – Step 2c

This section explains how dam break and flood routing may be undertaken in different ways and to different degrees of certainty. In addition to the fundamental methods for predicting dam breach and the consequent flood routing, consideration is also given to the selection of modelling scenarios, adequacy of data and existing sources of information.

19.1 Definitions

The prediction of dam break conditions is an important component of a risk assessment, since the predicted rate of release of water and the subsequent prediction of inundation area, along with flood conditions (that is, flow depth and velocity) all directly affect the potential for loss of life and damage. Since dams can fail in a multitude of ways and the hydraulic conditions that arise during dam break are very extreme; there can be relatively large uncertainties within the predictions compared with, say, the prediction of natural flood conditions.

While there may be recorded or observation data from natural flood events against which flood predictions may be validated, dam break conditions are typically far more severe and hence normally beyond the range of conditions for which historic data exist that may be used to validate the predictions. As such, the effect of uncertainty on the overall risk analysis should be considered.

There are three main aspects to predicting dam break conditions:

1) Predicting the breaching process and hence the flood hydrograph at the dam (that is, breach prediction)
2) Predicting how the flood hydrograph travels downstream, resulting in flood inundation (that is, flood routing)
3) Predicting the flood inundation that would be present if the dam did not fail, so the incremental impact of dam failure can be assessed

19.2 Key issues/concepts

Flood conditions downstream of the dam arising from a natural 100-year event may also be used as the basis for determining the extent of analysis for dam break assessment (that is, continue dam break flood routing until conditions match those expected during a 100-year natural flood event).

19.2.1 Adequacy of data

Where existing flood models and/or inundation maps are available, these may be used instead of recalculation, depending upon the tier of analysis being undertaken. Use of more detailed or accurate data than required for a given tier is acceptable. Since many reservoir owners/undertakers will have already undertaken some form of dam break assessment in the past, it is likely that these maps would be sufficient to support (at least) a Tier 1 analysis.

When undertaking analysis for a given tier, it is important to consider the source and accuracy of the data that you are using and whether they are appropriate for that level of analysis. For example, undertaking a Tier 1 inundation assessment by judgement can be made using 1:25,000 or 1:10,000 Ordnance Survey contoured plans. The purpose of the assessment is to rapidly determine the potential extent of inundation and hence to identify whether there is significant risk associated with the dam or not by identifying if people and property could fall within the inundation area. Prediction of flood levels in this way would be to within a few metres accuracy at best, with the accuracy diminishing the further you go from the dam, and hence the use of maps with reasonably spaced contours would be acceptable.

Reservoir inundation mapping (RIM) maps (Environment Agency 2009a) are available for many reservoirs in England and Wales, and can be used as part of a Tier 1 analysis. More detail on the RIM mapping is given in section 19.4.1.

19.3 Basis of tools in tiered approach

19.3.1 Breach prediction

Methods for breach prediction vary according to the type of dam structure and range from simple, regression analysis equations (based upon historic failures) to detailed numerical models. Recommended approaches for the different tiers of analysis are summarised in Table 19.1. The methods are explained in more detail in the following sub-sections, which also include guidance on how to predict breach as a result of cascade failure.

Table 19.1 Methods for predicting breach for different types of dam

	Tier 1	Tier 2	Tier 3
Earth	Use existing inundation maps where available OR Visual assessment assuming initial water depth half the dam height, and follow map contours/valley slope (with the option to refine using breach guidance from CIRIA Report C542 (Hewlett et al. 2000)	Use of Froehlich (1995) peak discharge equation combined with hydrograph estimation using the CIRIA Report C542 methodology. The prediction may be refined using soil erodibility and reservoir area data to guide on the likely nature of the flood hydrograph (Morris, 2013)	Use of simple rapid breach prediction model (FRMRC AREBA model (van Damme et al. 2011) for entry level analysis OR Use of numerical breach growth prediction model (for example, HR BREACH and WinDAM)
Concrete (thin)	Use existing inundation maps where available OR Visual assessment assuming initial water	Use of the CIRIA Report C542 methodology	Use of the CIRIA Report C542 guidance on potential breach size and rate combined with numerical modelling of discharge

	Tier 1	Tier 2	Tier 3
	depth half the dam height, and follow map contours/valley slope (with the option to refine using breach guidance from CIRIA Report C542)		OR Structural analysis, combined with hydraulic flow model
Concrete (mass gravity)	Use existing inundation maps where available OR Visual assessment assuming initial water depth half the dam height, and follow map contours/valley slope (with the option to refine using breach guidance from CIRIA Report C542)	Use of the CIRIA Report C542 methodology	Use of the CIRIA Report C542 guidance on potential breach size and rate combined with numerical modelling of discharge OR Structural analysis, combined with hydraulic flow model
Masonry	Use existing inundation maps where available OR Visual assessment assuming initial water depth half the dam height, and follow map contours/valley slope (with the option to refine using breach guidance from CIRIA Report C542)	Use of the CIRIA Report C542 methodology	Use of the CIRIA Report C542 guidance on potential breach size and rate combined with numerical modelling of discharge OR Structural analysis, combined with hydraulic flow model
Service reservoir	Use existing inundation maps where available OR Visual assessment and judgement[1]	Visual assessment and judgement[1]	Visual assessment and judgement[1] for breach size and rate combined with numerical modelling of discharge OR Structural analysis, combined with hydraulic flow model
Rules for assessing cascade failure	Use existing maps where available OR Visual assessment assuming initial water	Use existing maps where available OR Calculate subsequent cascade breach	Use of simple rapid breach prediction model (FRMRC AREBA model) for entry level analysis

	Tier 1	Tier 2	Tier 3
	depth is half the dam height,[2] and follow map contours/valley slope. Assume depth of water in reservoir is raised by the total volume released from the previous dam up to a maximum of 1m on the dam crest and consider the combined volume of water released when judging inundation extents downstream.	outflows by use of Froehlich (1995), but with the reservoir volume equally to the combined volume of the cascading flow and the reservoir retained water level calculated to reflect this volume of water stored within the reservoir to a maximum level of 1m above the crest.	OR Use of numerical breach growth prediction model (for example, HR BREACH and WinDAM) Each of these models predicts the breach initiation and growth process, hence when linked with dynamic flow models automatically predict conditions that would occur during cascade failure.

Notes: [1]Assessing the way in which a service reservoir may breach requires an assessment of how the structure could fail. In many situations the reservoir is constructed from a mixture of reinforced concrete surrounded by earth. Under these circumstances it is difficult to see how failure could occur quickly; the earth would need to be eroded before a significant block failure could occur. Since each design may be different, judgement is required to determine potential mechanisms and in particular the speed with which such as failure could occur. A likely upper bound would be to consider simple weir flow out of the reservoir across one panel width, using the depth of water in the reservoir as potential head.

[2]The uncertainty associated with judging potential water height at the time of dam failure is large, and probably greater than the additional depth that would arise as a result of cascade failure. For example, overtopping of an earth dam by 0.5–1.0m is likely to cause failure. On the basis of judgement, this would add perhaps 0.5m to the estimated depth of dam break flow downstream of the dam. It becomes more important to consider the combined volume of water released from the two or more reservoirs and how this would spread downstream than the more accurate prediction of flood levels very close to the dam.

19.3.2 Flood routing

Methods for predicting flood conditions arising from a breach are more widely available than for breach prediction (Table 19.2). Hydraulic conditions that occur during dam break can be very extreme – far more so than for natural flood events. Accurate prediction of these conditions requires the use of numerical modelling codes that can account for rapid transitions between super- and sub-critical flow along with the generation of shock waves. Simpler modelling packages or assumptions introduce greater uncertainty within the prediction and particular care should be taken in using flow depth and velocity data extracted from such simplified model analyses.

Base flow conditions within the downstream inundation area can significantly affect predicted flood conditions, depending upon the nature of the failure mechanism, size of catchment/river valley, volume of water retained and so on. For Tier 1 and 2 analyses it would be appropriate to ignore the river channel carrying capacity and simply take account of the overall valley shape. For Tier 1, 'full' reservoir conditions are assumed at failure; for Tier 2, dam critical conditions are estimated and associated to a probability of load event. For Tier 3 analyses, different downstream valley conditions may be considered as part of different hazard scenarios such as sunny day failure with low or normal river water levels, PMF failure with extreme flood conditions already within the valley, and combinations between these bounds. The choice of the different permutations will depend on the site-specific design and location.

Table 19.2 Methods for flood routing – all dam types

	Tier 1	Tier 2	Tier 3
Routing method – single dam failure	Use existing maps where available OR Visual assessment assuming initial water depth is half the dam height, and follow map contours/valley slope. Use judgement to assess potential distance inundated downstream from the dam.	Use existing maps where available OR CIRIA C542 (Hewlett et al. 2000) rapid routing method, updated by the Interim Guide (Brown and Gosden 2004), noting uncertainty within the prediction	Use of rigorous 2D flow models suitable for simulating extreme and rapidly varying flow conditions. Consider possible event trains and sensitivity to modelling parameters.

19.4 Analysis methods

19.4.1 Reservoir Flood Maps (RIM) maps

The Reservoir Inundation Mapping programme (Environment Agency 2009a) was undertaken by Defra in 2009 in order to provide indicative inundation maps for reservoirs to support emergency planning. The maps are intended to provide a conservative estimate of potential inundation in the event of dam failure. Maps can be accessed online through the Environment Agency website (www.environment-agency.gov.uk).

The maps were developed using simplified modelling techniques combined with predefined failure scenarios. The dam is assumed to have failed when overflowing depth reaches 0.5m above the crest level (or 0.1m for a non- impounding reservoir). In the event of a cascade failure, the same rules apply plus the timing of the second dam failure is such that the released flood wave coincides with the peak of the flood wave from the first. However, for cascades with three or more dams, the timing of each subsequent dam failure is not coincident with the peak of the arriving flood wave, hence does not offer an extreme upper bound to potential flood conditions.

The flood hydrograph from dam failure is calculated using the Froehlich (1995) equation with the flood hydrograph being generated through application of the CIRIA Report C542 methodology (Hewlett et al. 2000). As such, they contain a large degree of uncertainty and were developed only for use to support emergency planning. More detailed analysis should be undertaken where other uses are envisaged or clarification of flood risk at any particular location is sought.

Due to security concerns, detailed mapping information is restricted. Hence, details of potential flow depths and velocities are not available online. Even if these data were available, the simplified modelling techniques used will only provide indicative data with significant uncertainties in the values provided. However, as a source of information for undertaking an initial, Tier 1 assessment, the RIM maps might be used to support a visual assessment of potential flood conditions.

19.4.2 Visual assessment

A visual assessment is the simplest approach to estimating potential flood conditions that might arise if the dam failed. This entails use of judgement regarding the dam and mode of failure, volume of water released, and how this might spread across the local topography and further downstream. While a local estimation might be relatively straightforward, the accuracy of estimate will diminish as you travel further away from the dam and it becomes harder to estimate the rate at which flood wave attenuation occurs. Use of maps with contouring will help, but the process becomes increasingly uncertain away from the immediate dam area.

Where a dam might fail quickly, an assumption of water depth downstream roughly half the height of the retained water depth would be a conservative estimate. Where the dam is unlikely to fail quickly or completely, alternative, less extreme estimates can be made using judgement and taking into consideration the dam design and possible failure modes.

19.5 Additional guidance and limitations

CIRIA Report C542 (Hewlett et al. 2000) provided recommendations for estimating breach through earth and concrete dams, and a technique for the rapid calculation of potential inundation areas downstream without the need for numerical flow modelling. This was subsequently adapted for use within the *Interim Guide to Quantitative Risk Assessment for UK Reservoirs* (Brown and Gosden 2004). However, limitations on use of this method should be recognised. In Appendix 8 of CIRIA Report C542 it states that the method provides an approximate ‘rule of thumb’ method to estimate attenuation of a dam break flood wave. It also states that the method ‘should not be taken as a substitute for dam break flood simulation where it is important to quantify the actual risk to life and property in the downstream valley.’

The estimation of breach conditions through earth dams uses breach equations produced by Froehlich (1995). The failure of concrete or masonry dams uses guidance from CIRIA Report C542.

19.5.1 Simple rapid breach prediction models

As part of the Flood Risk Management Research Consortium (FRMRC2) research programme (www.floodrisk.org.uk) a new, rapid, simplified breach model, called AREBA, has been developed (van Damme et al. 2011). This model simulates breach formation through a homogeneous earth embankment. AREBA requires the user to define upstream load conditions along with an estimate of soil erodibility based upon soil type and state; the model then predicts breach growth and provides an estimate of the flood hydrograph. The model simulation is very fast – taking less than 0.5s to run –

allowing the user to play with the modelling parameters in order to understand the likely range of possible flood hydrographs that could occur.

AREBA simulates breach through homogeneous embankments, but does not simulate conditions through more complex, zoned or composite structures. The model simulates failure via seepage through (piping) or overflowing. Both surface erosion and headcut processes can be simulated. The headcut process is simulated using the methodology developed by the USDA for the SIMBA model.

The SIMBA model was developed by the USDA Agricultural Research Service, Hydraulic Engineering Research Unit (USDA–ARS–HERU) in Stillwater, Oklahoma. SIMBA is a simplified model, which simulates breach growth via headcutting erosion through cohesive earth embankments. As with AREBA, the breach growth process is predefined and applicable to simple, homogeneous embankments. SIMBA is the numerical breach model that sits within the USDA WinDAM software package (USDA NRCS 2012). The WinDAM code allows the user to also simulate the performance of grass cover and reservoir drawdown as a breach occurs.

19.5.2 Predictive breach models

Where a more refined estimate of breach conditions and the potential flood hydrograph is required and/or a more complex structure requires analysis, a more complex predictive breach model may be used. An example of such a model is the HR BREACH model, which allows simulation of breach formation through composite and zoned structures.

In 2009, a version of the HR BREACH model was integrated within the InfoWorksRS flow modelling package. This provides a truly integrated predictive breach model and a one-dimensional (1D) or two-dimensional (2D) dynamic flow modelling package, designed to be directly applicable for dam break analysis. Integration of the breach and flow analysis becomes particularly important when downstream flood conditions could affect flow through the breach and hence the rate of breach growth.

In 2010 the Centre for Energy Advancement through Technological Innovation (CEATI) facilitated Dam Safety Interest Group (DSIG) concluded a project which reviewed and evaluated the performance of breach models suitable for industry use and uptake. The international review concluded that the USDA SIMBA model and the HR Wallingford HR BREACH model offered the greatest potential for industry use. A new, merged version of the two models was proposed.

Continued development of the ‘standalone’ HR BREACH model has resulted in a version that can now simulate breach through zoned structures – structures with multiple types or states of material, such as cores or berms. It has been demonstrated that different zones of erodibility can significantly affect the breach outflow characteristics (Morris, 2013). Hence where a more certain estimate of breach conditions is required for a zoned structure, such a modelling approach offers the best solution (as of 2013).

19.5.3 Structural analysis

While breach models have been developed to predict erosion through earth structures, no such models have been developed to simulate the failure of concrete or masonry structures. Consequently, breach prediction for such structures relies on one of the following:

- expert judgement based upon how the structure might fail
- simple guidance from CIRIA Report C542 (Hewlett et al. 2000), which suggests a percentage area of structure failure and rate
- structural analysis of the dam to provide information on how the dam might fail

A better estimate of potential flood conditions can be gained by using hydraulic models to simulate flood flow through the dam according to breach growth instructions defined by the user (that is, to simulate the rate and size of breach growth according to expert judgement).

19.5.4 Flood routing

A number of modelling approaches may be used to predict potential inundation outlines, flows, depths and velocities. In the 1980s and 1990s, these models tended to be 1D flow averaged models, solving the St Venant equations (for example, DAMBRK). In the late 1990s and 2000s, 2D flow models became more widely available and used, in particular supported by the development of GIS packages to manipulate topographic and mapping data. In the early to mid 2000s, a number of simplified, rapid 2D models were developed to cope with modelling and mapping large areas.

Use of 1D flow models is now becoming less practical, since topographic and mapping data can be easily handled within GIS packages. Hence it becomes more economical to undertake 2D modelling, or linked 1D and 2D modelling and mapping. Two-dimensional modelling has the advantage of not requiring the user to predefine the flood flow path. Hence, full 2D dynamic solutions of the St Venant equations offer a good tool for modelling dam break conditions.

The relatively recent development of simplified 2D flow models provides a tool that can rapidly map potential flooding for large areas, but to a lesser accuracy. The models do not provide a complete solution to the St Venant equations; by missing some of the terms (for example, flow momentum) the equations can be solved far more quickly. Consequently, the accuracy of such modelling is less than that achieved with the more rigorous 2D flow models and care should be taken, particularly in using flow depth and velocity data for impact analyses. The accuracy of such data will vary according to site specific conditions.

In 2009 the Environment Agency undertook a review of the performance of different 2D flow models (Environment Agency 2009b). The subsequent report provides recommendations as to when different types of model might be used and an assessment of the performance of different commercial models at that time.

20 Consequence analysis

This supporting information in respect of consequence assessment comprises additional guidance and where appropriate sources used for methodology and developments thereof, subdivided into the various elements of a consequence assessment. It is preceded by a short description of the basis of the system incorporated in the guide.

20.1 Key issues and concepts

20.1.1 The basis of the consequence methodology

In order to ascertain the impact of dam failure the potential receptors need to be identified. The receptors that could form part of an impact assessment are outlined in Table 20.4. The level of detail will vary with Tier 1, 2 and 3.

Basis for Tier 1 consequences

The Tier 1 (qualitative) method is a simple matrix-based system, with the adopted method being based on specific receptors under the following key impact category headings:

- people
- economic activity
- the environment
- cultural heritage

However, key receptors within these categories, other than those given in Tier 1, could be considered in a similar way provided five consequence magnitude classes can be distinguished. Many sub-categories are listed in Table 20.3.

Key elements of the Tier 1 (qualitative) method are summarised in Table 20.1.

Table 20.1 Key elements of Tier 1

Issue	Basis of methodology used in Tier 1
Output matrix of Consequence magnitude	Table 4 of ANCOLD draft guidelines (ANCOLD 2011), but amended (a) from seven consequences classes to five and (b) to use number of properties at risk, with the number derived assuming fatality rates of 7% and 2% for moderate and extreme hazard respectively (that is, as given in FD2321; Defra and Environment Agency 2006), thus giving boundaries between each class average societal life loss (ASLL) of 0, 0.1, 1, 10 and >10,
Data on people at risk	Ordnance Survey 1:25,000 scale maps
Property damage	Not explicitly included in matrix, as included by virtue of considering risk to lives of people assumed to be present in houses

Basis for Tiers 2 and 3 consequences

The Tier 2 methodology balances the degree of simplification of the analysis with the effort required to produce more accurate answers. This is exemplified by the consideration of the number of scenarios that could be contemplated. Key elements of the analysis are summarised in Table 20.2.

Table 20.2 Key elements of Tier 2

Issue	Basis of methodology used in Tier 2
Data on people and property at risk	Ordnance Survey 1:25,000 scale maps
Fatality rate	Adapted from Reclamation (1999)
Property values	FHRC (2010), augmented with property write-off values from ODPM (2004) datasets

The tools suggested in section 20.3 are the best currently available international methods for Tier 3 consequence analysis.

20.1.2 Defining consequence (population) scenarios

Table 20.3 describes some consequence scenarios that might be considered considering the population downstream of the dam. On the basis of this table, where there are two dams in a cascade, the number of possible combinations of circumstances that may occur and thus the PAR/LLOL combinations to be considered could be 36! This demonstrates the need to consider carefully the choice and number of scenarios to be included in the risk assessment.

Table 20.3 Possible combinations of dam break and downstream population scenarios

	Scenario	Number of choices	Choices	Remarks
1	Type of failure	2	(a) Sunny day failure (b) Rainy day failure	As given in Table 7.4
2	Warning time prior to breach	Range		Where a warning can be given to downstream communities and individuals, the loss of life should be significantly reduced.
3	Time from onset of failure to peak breach discharge	Range		Shorter time likely to give higher peak flow and thus greater depth of flooding and velocity

4	Number of dams	2+	(a) Subject dam only (b) Dam(s) in cascade upstream and/or downstream of subject dam	Volume of the dam breach will include the volume of the other reservoirs in the cascade, for example, triggering the cascade failure and/or whose failure is triggered by the subject dam.
5	Antecedent flow downstream	2	(a) Sunny day on adjacent catchments (b) Extreme rainfall on adjacent catchments as well as subject catchment	(a) Pre-breach flow in the downstream river small for (a) and large for (b), so the maximum flood levels will be higher for (b) (although the incremental damage may be less) (b) For (b) the rainfall on the adjacent catchment could be the same return period as that on the subject dam, or some lesser value (for example, 1,000 years), or greater.
6	Time of day (and day of week)	3	(a) At night when most people are asleep at home (b) In day when most people are at work (c) Evenings and weekend, when most people are shopping or at recreation sites	The population at risk may vary significantly with time of day and day of the week; for example, if the majority of installations downstream were non-residential properties.
7	Nature of population	Range	Range from infirm and elderly/infants to fit young adults	R2P2 and ANCOLD use the concept of 'a statistical life'; to avoid judgements about the value of different types of population.

20.1.3 Identification of consequences

Key receptors are listed in Table 20.4.

Table 20.4 Key receptors potentially at risk during a breach event

Potential receptor	Description
Residential properties	All residential properties including upper floor flats in total destruction zone
Non-residential	All non-residential properties categorised into general use

Potential receptor	Description
properties	(office, retail, warehouse, factory or public)
Susceptible properties	Properties with vulnerable patrons including schools, health centres, residential homes and prisons
Key service properties	Properties whose use will aid recovery during an event includes police, ambulance, fire
Key utilities	Major facilities such as power stations, roads and railways (including bridges), gas mains, telecommunications cables, electricity substations
Undertaker asset	All assets belonging to the undertaker
Life	Death during event
Construction/ decommission	Reinstatement of asset or decommission and works to make site safe
Impact on business	• Loss incurred by company (transfer of loss will be accounted for) • Nationally important manufacturing may need specific consideration • Third party costs for business disruption
Emergency service	Reactionary activities during and event
Local utilities	Smaller asserts such as sub-stations
Use of water by undertaker	Impact of loss of asset of set period.
Transport	Construction/replacement of asset and cost of loss/closure of asset during event
Development areas	Areas of both green field and brown field land currently allocated for future development
Temporary accommodation	Housing residents when buildings are uninhabitable
Health	Injury, illness and stress
Environmental	Nationally important features such as SSSI and Biodiversity Action Plan (BAP)
Ecological	Impact on local features and habitats
Historical	Loss of historical monuments, artefacts or features
Loss of service	Local service such as schools, police, health
Motor vehicles	Write-off of vehicles in area
Recreation	Loss of facility for local population
Agriculture	Loss of crop or livestock
Fines, claims and legal	Associated with responsibility for the breach

Methods for analysing the secondary or indirect effects resulting from flooding that potentially pose a risk to life (for example, gas main explosion, accident on traffic diversion) are not considered in this guidance but should be considered on merit.

Available datasets for receptor identification are listed in Table 20.5.

Table 20.5 Resources for identifying receptors and consequences

Resource	Source
OS mapping	Ordnance Survey
OS MasterMap	Ordnance Survey
Flood inundation maps	Undertaker Environment Agency
National Property Dataset	Environment Agency
Residential valuations	Land Registry
Non-residential valuations	UK Statistics
Undertaker assets	Undertaker
Agricultural land use	www.magic.gov.uk
Motorway use statistics	Highways Agency
Railway use statistics	Network Rail
Ancient woodland	www.gis.naturalengland.org.uk
Area of Outstanding Natural Beauty	www.gis.naturalengland.org.uk
Battlefields	www.gis.naturalengland.org.uk
Country parks	www.gis.naturalengland.org.uk
Heritage coast	www.gis.naturalengland.org.uk
International bird areas	www.rspb.org.uk
Listed buildings	http://list.english-heritage.org.uk/mapsearch.aspx
Local Nature Reserves	www.gis.naturalengland.org.uk
National Parks	www.gis.naturalengland.org.uk
National Nature Reserves	www.gis.naturalengland.org.uk
Registered parks and gardens	http://list.english-heritage.org.uk/mapsearch.aspx
Ramsar sites	www.gis.naturalengland.org.uk
RSPB reserves	www.magic.gov.uk
Special Areas of Conservation	www.gis.naturalengland.org.uk

Resource	Source
Scheduled monuments	http://list.english-heritage.org.uk/mapsearch.aspx
Special Protection Areas	www.gis.naturalengland.org.uk
Sites of Special Scientific Interest	www.gis.naturalengland.org.uk
Undetermined grassland BAP	www.gis.naturalengland.org.uk
World Heritage Sites	http://list.english-heritage.org.uk/mapsearch.aspx

20.1.4 Subdivision of level of impacts

The severity of the impact varies by several orders of magnitude from shallow flooding less than 0.5m deep with low risk to life to water above the tops of single storeys buildings such as bungalows which is flowing fast enough (and/or carrying debris) to wash the buildings away.

Although this can be taken into account at Tier 2 by considering Q/W (or VD) this is not possible at Tier 1. For simplicity we have not subdivided into impact classes except for impacts to people based on height of the dam (neglected effect of distance downstream). The height is selected on the following basis.

- Use VD of >3.m^3/s/m, the value for onset of partial structural damage (equivalent to fatality rate of 5%).
- Assume dam break flow is say five times this (expansion from width of breach to width of valley), so use dam break discharge of 15m^3/s/m as boundary between two levels of impact in Tier 1.

This is equivalent to dam height of 5m above the flood plain (unit discharge in dam break considered as broad crested weir)

20.2 Guidance for consequence assessment – Step 2d

20.2.1 Impact on human health and life

The measures of impact on people are summarised in Table 20.6. It should be noted that normally only a hypothetical person is considered, a hypothetical person being defined in Appendix 1 of R2P2 (HSE 2000) and comprising 'for each population exposed to the hazard, there will usually be a hypothetical person specifically constructed for determining the control measures necessary to protect that population.'

Table 20.6 Measures of impact on people

Measure	Definition	Comment
Population at risk (PAR)	Those in the inundated zone in the event of dam failure, if they take no action to evacuate	Forms the basis for planning evacuation. Neglects the fact that some people would not be at direct risk, for example, if in upper storeys of buildings above floodwater, where floodwater does not pose a threat to structure of building
Highest individual risk (HIR)	Highest product of risk of death to an individual and exposure of that individual (expressed as % of time over 24 hours/ 365 days they are present)	One of the criteria used to assess whether risk is tolerable
Average societal life loss (ASLL)	ASLL is the sum of the product of PAR and individual fatality rates for each of the population groups in the inundated area, for example, if a population group of two people has a 5% chance of death in the event of dam failure ASLL = $2 \times 0.05 = 0.1$.	A second criterion to assess whether risk is tolerable
Risk of injury to people	Number of injuries within a particular hazard 'zone' given the number of people within the hazard zone (at ground level), the flood depth/velocity and debris factor), the effectiveness of flood warning, the speed of onset of flooding and nature of area, and the vulnerability of those present who are very old and/or infirm/disabled/long-term sick)	FD2321 (Defra and Environment Agency 2006) gives a methodology to estimate the number of injuries

Population present in inundated area

Guidance on estimating typical numbers of people in a potentially inundation area is given in the methodology in Part 1 for Tier 2, with supplementary information below.

Non-residential

The occupancy factor for people in non-residential properties varies very significantly depending on the use of the building, with use specific values given in Table 20.7. For a small number of properties at risk, or where an individual property has a major effect on outcomes from the risk assessment, property specific assessment may be appropriate. However, where there are a large number of properties are at risk, average values are likely to be reasonably representative.

Where property-specific values are required, as well as consideration of the property use, an alternative check on the number present may be made by using the area of car parking as an independent check on building occupancy, where the area of car parking, including access lanes to bays, is typically about $25m^2$/ car. Where most occupants will have travelled there by car, the number of occupants is likely to be broadly equal to the number of car spaces; this is reduced where car sharing, walking to the property and public transport are used to travel to the building.

Defining groups of people/property

A key part of the analysis is the subdivision of the inundated area into appropriate groups for the analysis of hazard from the dam break flood. This subdivision will depend on factors including:

- whether dam break analysis is 1D (when velocity is average across a section) or 2D, when both depth and velocity vary across the inundated area
- the quality of the ground model, and whether thresholds of property are available (or they are assumed to be a common height above ground level, such as 150mm), that is, to what extent corrections can be made for reduced depth of flooding for properties near the edges of the flood plain

It is normal to accept some form of averaging – increasing with distance downstream when the potential effects of the dam break reduce.

Exposure (distribution of the population)

Careful consideration should be given to the number of scenarios to be used in the analysis. One approach is to consider a time averaged approach, namely the sum of the number present over a particular period, multiplied by the duration as a percentage of time over 24 hours/365 days in a year, as shown in Box 20.1.

Table 20.7 Values of occupant area and occupancy factor in non-residential properties

MCM code	Type of property	Typical area for internal circulation and activity (Pickard 2002)		Occupant area (m^2 per person) in Table 4.1 of CIBSE (2003)		Suggested normal value	
		m^2 per person	Page	UK	US	Occupant area[1] m^2/ person	Occupancy factor % of hours/ year
–	**Average for all non-residential**			na	na	40	25%[4]
	ODPM bulk						
21	Retail	Out of town[2]: food retail 14 non-food retail; 20	342	2.0–7.0	2.8–5.6	30	30%
23	Service industries	15 to 30	201	na	Na	40	21%
3	Offices	15-20	286	6	9.3	40	21%
410	Warehouses	Not given	207			200	30%
8	Factories (workshops)	28	201	5	9.3	60	21%
	Other						
214	Distributive trades (builders merchants and so on)	80	201	na	Na	160	21%
22	Garages	na	na	na	Na	160	21%
234	Public houses	na	na	na	Na	10	15%
235	Restaurants	1.3–1.9 excl. kitchen and so on (which is $2m^2$/cover)		na	Na	8	10%
511	Hotels	28–75 (1–5 star)	145	na	Na	100[5]	50%
610	Schools	Primary 5; Secondary 8[3]	48, 54	na	Na	7	20%
630	Assembly halls	0.85–1.0 seating area (excl. public areas)	20	0.5	1.4	5	5%
810	Farm buildings			na	Na	1/ building	30%

Notes [1] Broadly double the value suggested for design of new buildings, to allow for less efficient use of older buildings and some empty buildings

[2] Based on car parking maximum standard in PP6; assuming 50% for ancillary accommodation is compensated for by shop staff

[3] Building Bulletins 98 and 99 (Briefing Framework for Primary School Projects (2005) and Secondary School Projects (2004) respectively) suggest gross areas in m^2 is 340 + 4.5N for primary schools, 1000+5.4N for 8-12 middle schools, and 2250 +7N for 11-18 secondary schools where N is the number on the roll.

[4] Provision for working week, plus time in recreational non-residential buildings (sports facilities, pubs and so on).Ground floor may comprise restaurant, bar (public house).

na = not applicable; ODPM = Office of the Deputy Prime Minister

Box 20.1 Example of calculation of time averaged population

The time-averaged population is 'the population associated with each possible location for each scenario' times the 'proportion of time that each scenario represents, as a percentage of the time in a year'.

An example: A mid-range Premiership football stadium might expect to be used on 25 occasions in a year for about four hours each time with an average attendance of 35,000. The probability of the stadium being occupied at any time through the year is (4/24) × (25/365) = 0.0114 (1.1%)

The average population at risk could thus be said to be 35,000 × 0.0114 = 399.

Because the car parking area is in the stadium grounds and the crowd has come and gone from the vicinity of the stadium within the four-hour window on each occasion, they do not need to be taken into account separately while in the car park.

In addition there may be say 20 persons who normally work at the grounds, giving a further PAR of: 20 × 8 hours × 5 days/ (24 hours × 7 days) = 4.8 average.

The total population at risk on a 24 hour/ 365 days/ year would therefore be taken as 404. The risk, expressed as probability × consequences, would then be P 404 = 404P.

This is the approach used in risk based assessments of planning applications near major hazards by HSE (Carter 1985), with quoted values as shown in Table 20.8.

Table 20.8 Types of hypothetical person and associated average exposure time, as used by HSE in evaluating planning applications near high hazards

Hypothetical person	Exposure (that is, part of year present)		Justification
	Used by HSE for development (Carter 1995)	Suggested for existing dams	
House occupant	100%	80%	Mother with pre-school children, some elderly and/or unemployed
Hotels	100%	70% for small, 85% where large used for conferences	Occupant may have minor illness Staff who live and work in hotel
Hospitals and nursing homes	100%	As Carter (1995)	
Factories	75%	55%	Factor greater than likely occupancy because of enhanced risk as processes in the factory may be adversely affected by the flood leading to additional fatalities 55% allows for single shift and x2 for processes. Increase to 100% for 24 hour working

Hypothetical person	Exposure (that is, part of year present)		Justification
	Used by HSE for development (Carter 1995)	**Suggested for existing dams**	
Places of entertainment	50%	As Carter (1995)	Enhanced factor to reflect unfamiliarity of occupants with building and its exits leading to increased risk
Shops and supermarkets	50%	As Carter (1995)	
Warehouses	50%	As Carter (1995)	Flammable materials or other substances may be stored which would increase the fatality rate
Office worker	30%		50 hours per week, 52 weeks per year
School	25%		Staff/ caretaker
Sunday markets, car boot sales	7.5%		
Camper		40%	Assume campers present 16 hours/ day, 8 months/ year
National trail		65%	Assume people on trail most of daylight hours

Source: Carter (1995)

Alternatively the population distribution scenarios could be broken down into specific periods, for example, a short period when a large population is present such as at a music festival or football club. In this case the probability of failure will need to take into account the duration of time that the increased population is present, such that instead of an annual probability it would be annual probability reduced in proportion to the reduced presence, over a year. The sum of all durations by which annual probability is multiplied must equal 1. This is illustrated in Box 20.2.

Box 20.2 Example of calculation of short-term population

Example: A mid-range Premiership football stadium might expect to be used on 25 occasions in a year for about four hours each time with an average attendance of 35,000. The total population at risk for the duration of the event would therefore be taken as 35,000.

However, the probability of failure would need to be reduced for the reduced duration, that is, multiplied by $(4/24) \times (25/365) = 0.0114$ (1.1%).

The risk, expressed as probability x consequences would then be $P \times 0.0114 \times 35{,}000 = 399P$.

This approach is then only likely to give different outcomes from a time-averaged approach to population where the probability of dam failure varies with time over a year, such that the downstream population is increased at times of increased probability of failure of the dam.

Threshold of flood hazard to people

Historically the population at risk was taken as (Binnie & Partners 1991) the population in the areas where both the product of depth and velocity is greater than $0.5m^2/s$, and the depth above external ground level is greater than 0.5m.

Fatality rate

There are a variety of models to estimate life loss due to dam failure, with a good summary given in Aboeleta and Bowles (2005) in their draft report to ANCOLD and the US Army Corps of Engineers in LIFESim. Other models include RESDAM (Maijala 2001) and Li et al. (2012). Data required include (Bowles et al. 2008, ASDSO 1988):

- topography
- time varied 1D or 2D inundation maps,
- GIS property database with number (and types) of structures
- population data from the Census
- means of evacuation – vehicle, sport-utility vehicle (SUV), pedestrian
- road network
- permanence of buildings/ resistance to flood depth and velocity

With more sophisticated methods of estimating fatality rates, the following factors are included in the analysis:

- role of shelters (buildings or other physical obstructions) in reducing life loss
- warning
- human reaction to warning, or rising floodwater
- permanence/ resistance of buildings
- traffic congestion in trying to quickly evacuate large urban areas

The method adopted in the guide for Tier 2 is based on the Reclamation paper DSO-99-06 (Graham 1999), with evidence reproduced in Part 2 with the guidance on Tier 2. This is an averaging approach, based on observed fatality rates in recoded dam failures and flash floods. This implicitly allows for, although not quantifying, most of the above in order to permit a simple quantitative risk assessment.

FD2321 (Defra and Environment Agency 2006) provides an alternative methodology to estimate fatality rate, namely 2 × hazard rating, although the evidence for this is not included in the report. At flood discharges of less than $10m^3/s/m$ this gives higher fatality rates, with a comparison given in Figure 7 of Brown et al. (2012). The reason for this is unknown, but is assumed to reflect consideration of people in the open rather than in shelters. Thus the method given in DSO-99-06 data was retained as being more applicable to dam break scenarios.

Effect of warning and shelter on fatality rate

In estimating the base case highest individual risk and average societal life loss it should be assumed that there is generally no warning. The exception is where the population at risk is well downstream of the dam with an intervening community where it may be reasonable to assume that the alarm would be raised once the flood wave had passed the first community and that the population downstream would be warned (allowing a reasonable time for the authorities to receive the alarm and issue warnings). Where allowance is made for some warning this should be stated in the impact assessment for the dam. It is considered unlikely that in the UK context any effective warning would be given.

Although shelter has an important influence on likely fatality rate, there is no commonly agreed simple method for taking this is into account. The averaged fatality rates reported implicitly account for shelter where these are derived from incidents of dam failure and these are normally used for Tier 2 analysis. If shelter is to be taken into account then it would be necessary to change to a Tier 3 analysis.

20.2.2 Economic damage

Level of damage to buildings

Binnie & Partners (1991) provided a good summary of estimating the damage potential in dam break floods, where buildings were destroyed when depth $\times$ velocity >7 m^2/s and inundation damage only occurs when dV >3m^2/s and partial structural damage in-between these two values.

This is an area of on-going research, particularly in respect of tsunami damage on buildings, and how detailing and design of buildings can reduce structural damage in event of impact by a flood wave (and the associated debris).

Values for the impact on transport infrastructure are given in the Highways Agency Design Manual Risk for Roads and Bridges (Highways Agency 2012).

Valuation of inundation damage to buildings

Suggested values for inundation damage are shown in Table 20.9, all values being for short duration (<12 hour) floods and national averages (rather than local data)

Note that the cost of inundation damage includes the cost of clean-up after contamination by substances in the flood water, which may include sewage, oil and other industrial liquids.

Table 20.9 Valuation of inundation damage of buildings

Sector average	Non-residential (£/m²)		Residential (£/house)
	Value	**Inundation damage**	**Inundation damage**
Building structure	£864		£23,300
Services	£400		
Moveable equipment	£140 (Note 2)		£20,800
Fixtures and fittings	£140 (Note 2)		(Contents)
Stock	£180		
Total (sector average)	£1,724	£881/m^2 at 3m depth flooding	£44,114/ property at 3m depth flooding
Source	Weighted average for 221, 234, 310, 410, 610, 810 (75% total)	Appendix 5.5, Weighted mean of all data	Sector average

Notes: [1] All values taken from December 2005 version of FHRC 2010
[2] Section 5.7.1 of FHRC 2005 notes this is set at 50% of replacement values

Cost of building destruction

There is no equivalent statistical analysis of the cost of building destruction, equivalent to the data on inundation data in the Multi-coloured Handbook (and summarised above).

Where a building is destroyed, one means of valuation is its market value, as if the owner/undertaker were paid this they could buy an equivalent property elsewhere.

An alternative means of valuation is to consider the direct and indirect costs which might include the following, and may exceed market value:

- administration costs covered by insurance company, such as lawyer's fee relating to negotiating compensation
- emergency accommodation
- demolition
- alternative accommodation while obtaining necessary approvals and rebuilding
- rebuilding, including professional fees
- for non-residential property lost income until building is functioning

The excess of market value over rebuilding costs (the land value) provides some allowance for those costs which would be additional to rebuilding cost.

It is therefore suggested that the market value of the property and its contents (which need to be added separately) is an appropriate means of valuation for estimating the consequences of failure for use in dam safety management. Application of this approach, as expanded below, is set out in Table 20.10.

For residential property it is suggested that the regional property price, available on the Land Registry website (www.landreg.org.uk) is used. As this is a quarterly average it may be appropriate to use the average value over a year.

Table 20.10 Preliminary valuation of cost of destruction of buildings

	Non-residential (£/m^2)		Residential (£/house)	
Building market value	**£600**	**Note 1**	**£191,300**	**UK average; from Land Register Oct–Dec 2005**
Contents (replacement value)				
Services	£400	As Table 10.1	£41,600	FHRC × 2 floors
Moveable equipment	£280			
Fixtures and fittings	£280			
Stock	£180			
Total (sector average)	£1,7400/m^2		£232,900	/ house

Notes: [1] Building value is as used for capping damages in damages assessment of flood alleviation schemes rather than value in FHRC 2010, namely 'capital value = 100/ equivalent yield × rateable value'. Above value based on rateable value of £54/m^2 (ODPM, 2004 value for UK) and yield of 9%.

Impacts on agricultural land

The cost of a reservoir breach on agriculture will depend on the type and productivity of land which is inundated. The impacts on otherwise well-drained improved grassland will be much greater than on poorly drained, extensive grassland, especially in the summer. Deep or turbulent flooding can cause extensive damage to standing crops.

In this guide only qualitative methods of assessment are used in Tiers 1 and 2 to estimate the area of agricultural land affected by Agricultural Land Class (ALC) type (see below). If potential impacts are thought to be significant, a quantitative method could be used by applying the FHRC method for the assessment of agricultural benefits. This method defines the agricultural productivity, impact of flooding and produces a monetary value for the cost of a single flood according to the ALC. Scenario II in FHRC 2010, (p. 158) details this method. However, this method should be used with caution as it includes assumptions about the duration of the inundation and also economic adjustment factors for the purposes of flood defence appraisal which may not be appropriate for the estimation of damages arising from reservoir flooding.

In more detailed assessments, considerations can also be given to the effects of dam failure on the loss of income subsidies paid to farmers and the high value of potato, vegetable, flower and fruit crops.

For these reasons application of quantitative analysis of agricultural damages is not advocated here for Tier 1 and 2 risk assessments.

Box 20.3 Agricultural Land Class (ALC) system

The ALC system classifies land into five grades, with Grade 3 subdivided into Subgrades 3a and 3b. The best and most versatile land is defined as Grades 1, 2 and 3a by policy guidance – see the *National Planning Policy Framework (2012)* or MAFF (1988). This is the land that is most flexible, productive and efficient in response to inputs and which can best deliver future crops for food and non-food uses such as biomass, fibres and pharmaceuticals. Current estimates are that Grades 1 and 2 together form about 21% of all farmland in England; Subgrade 3a contains a similar amount.

After the introduction of the ALC system in 1966 the whole of England and Wales was mapped from reconnaissance field surveys to provide general strategic guidance on land quality for planners. This provisional series of maps was published on an Ordnance Survey base at a scale of one inch to one mile in the period 1967 to 1974. These maps are not sufficiently accurate for use in assessment of individual fields or development sites, and should not be used other than as general guidance. They show only five grades: their preparation preceded the subdivision of Grade 3 and the refinement of criteria, which occurred after 1976. They have not been updated and are being allowed to go out of print. A 1:250 000 scale map series based on the same information is available. These are more appropriate for the strategic use originally intended. This data is now available on 'Magic', an interactive, geographical information website (www.magic.gov.uk).

Definition of ALC grades and subgrades

- **Grade 1 – excellent quality agricultural land**. Land with no or very minor limitations to agricultural use. A very wide range of agricultural and horticultural crops can be grown and commonly includes top fruit, soft fruit, salad crops and winter harvested vegetables. Yields are high and less variable than on land of lower quality.
- **Grade 2 – very good quality agricultural land.** Land with minor limitations which affect crop yield, cultivations or harvesting. A wide range of agricultural and horticultural crops can usually be grown but on some land in the grade there may be reduced flexibility due to difficulties with the production of the more demanding crops such as winter harvested vegetables and arable root crops. The level of yield is generally high but may be lower or more variable than Grade 1.
- **Grade 3 – good to moderate quality agricultural land**. Land with moderate limitations which affect the choice of crops, timing and type of cultivation, harvesting or the level of yield. Where more demanding crops are grown yields are generally lower or more variable than on land in Grades 1 and 2.
- **Subgrade 3a – good quality agricultural land**. Land capable of consistently producing moderate to high yields of a narrow range of arable crops, especially cereals, or moderate yields of a wide range of crops including cereals, grass, oilseed rape, potatoes, sugar beet and the less demanding horticultural crops.
- **Subgrade 3b – moderate quality agricultural land.** Land capable of producing moderate yields of a narrow range of crops, principally cereals and grass or lower yields of a wider range of crops or high yields of grass which can be grazed or harvested over most of the year.
- **Grade 4 – poor quality agricultural land**. Land with severe limitations which significantly restrict the range of crops and/or level of yields. It is mainly suited to grass with occasional arable crops (for example cereals and forage crops) the

yields of which are variable. In moist climates, yields of grass may be moderate to high but there may be difficulties in utilisation. The grade also includes very droughty arable land.

- **Grade 5 – very poor quality agricultural land.** Land with very severe limitations which restrict use to permanent pasture or rough grazing, except for occasional pioneer forage crops.

20.2.3 Other impacts

Impacts on the environment and cultural heritage

To avoid disproportionate time and resources being spent on environmental and cultural heritage benefit assessments, it is important to consider if there is an environmental or cultural heritage concern within the inundation area significant to warrant such time and resources in assessment. At Tier 2, for example, this can be limited to international and European designations.

It is suggested that these are normally recorded as a list of items, rather than trying to quantify a financial or economic loss. The ANCOLD guidelines (ANCOLD 2011) provide a rating system with four categories of damage (minor, medium, major and catastrophic) for each of infrastructure costs, health and social impacts, and environmental impacts. In this guidance a similar approach has been taken to gauge consequence magnitudes for transport, agriculture, the environment, and cultural heritage but, in addition, considers flood depth and velocity, or the percentage area affected (whichever is applicable).

Indirect costs

Where they could be significant, the cost of emergency services could be taken into account. These are normally taken as 10% of the property damage (FHRC 2010).

As well as direct physical damage there will be a wide variety of other impacts in the event of dam failure, with the Floods Directive (European Commission 2007) and Water Framework Directive (European Commission 2000) putting various requirements on government to assess the risk at a national level.

Reconstruction costs of dam

At feasibility level, an indicative cost estimate may be obtained by considering a notional embankment section with, say, a 4m wide crest, 3H:1V side slopes to derive a notional volume of fill and then using an all-in rate of £xx/m^3; this rate will vary with dam size and complexity, and should be established on a project-specific basis to get a construction cost. Where foundation conditions are poor, flatter slopes will be required, with the existing slopes providing a guideline value. Where more accurate values are required, a separate cost should be estimated for the different elements of the dam (for example, removal of existing dam, spillway, outlet and foundation treatment).

20.3 Economic/financial valuation

Current government guidance on economic appraisal is given in HM Treasury's Green Book (2003) and Defra's flood and coastal erosion risk management appraisal guidance (FCERM AG) (Defra 2003).

It is a matter for users whether they use 'market value' or 'rebuilding cost' (the latter is deemed to include demolition, refitting and restocking costs), with relevant factors noted below. Of relevance to the choice will be a view on:

- how insurance companies would be likely to value claims for compensation against the reservoir owner/undertaker
- how Ofwat (or other regulator of privatised companies) view benefit/cost calculations relating to reservoir safety

The differentiation between 'financial' and 'economic' loss is shown in Table 20.11.

Table 20.11 Comparison of economic and financial valuation of damages

Type	Definition	Comment
Financial	A loss to a supplier, which does not necessarily represent a loss to the UK economy if the consumer can purchase the same product from an alternative supplier within the UK.	Broadly replacement costs, and loss of revenue
Economic	Considered from the national perspective rather than the local. This means, for example, the loss of tourist revenue from the disruption or destruction of a tourist facility is not considered, only the replacement cost. The rationale for this is that the tourists can go to another venue within the UK, and as such, the revenue has still been generated within the UK with no economic loss.	Approximately depreciated costs, for example market value

When deciding on investment in flood defences (which are usually publicly owned), Defra (Flood Management) is typically concerned with potential benefits and costs to the national economy. Public funds are made available for their maintenance and improvement where this is shown to be in the national interest. Defra therefore uses economic loss in evaluating the potential flood damages from fluvial flooding.

By contrast, even though the consequences that arise from a dam failure may impinge on the public domain, the onus of responsibility for controlling and reducing the risk of failure of a dam usually rests with a private owner/undertaker. In this situation where third parties and their insurance companies are seeking compensation it is financial loss that will be valued.

The objective of the flood damage assessment is to help decide whether work should be done (and money should be spent) on reducing the risk of failure of a dam by carrying out structural or other improvements. In simple terms, the potential cost of flood damage avoided by undertaking that work (alongside the more important factor of the number of lives that may be preserved) is compared with the financial costs of work and thereby a decision is taken on whether to implement dam improvements. It is recommended therefore that a financial measure of loss is used in evaluating potential damage which would arise from dam failure.

20.3.1 Incremental or total damages

An important point is whether the consequences of dam failure are taken as total or incremental, relative to the no-dam failure scenario. This does not apply to the sunny day failure, only the rainy day scenario. Although in many situations it is satisfactory to consider only the total consequences, in some circumstances, such as small dams on large catchments, this can be very misleading as to the consequences of dam failure. This is a user decision, relative to the circumstances of the individual dam.

20.3.2 Advanced tools for calculating and modelling flood impacts

There are several bespoke tools available for the calculation of flood impacts and losses. These are typically used at a Tier 3 level of analysis.

Calculation of highest individual risk (HIR) can be achieved by selecting and applying one of the following methods:

- Reclamation method (Graham 1999)
- LIFESim
- HEC-FIA

Reclamation method (Graham 1999)

This method provides a suggested range of fatality factors that can be multiplied by the population at risk (PAR) to obtain an estimated life loss. The fatality factors are a function of the following three parameters that influenced life loss:

- flood severity (low, medium, or high)
- warning time defined the elapsed time between when the first official warning reaches the PAR and the arrival of the flood wave
- flood severity understanding (vague or precise) – the extent to which the PAR understands the severity of the approaching flood

LIFESim

LIFESim is a spatially distributed dynamic simulation modelling system for estimating potential life loss. It considers evacuation, detailed flood dynamics, loss of shelter and historically based life loss. LIFESim can be used to provide inputs for dam safety risk assessment and to explore options for improving the effectiveness of a dam owner's/undertaker's emergency plans or a local authority's response plans. Development of LIFESim was sponsored mainly by USACE and ANCOLD.

LIFESim was formulated using an underlying development philosophy that emphasises the inclusion of the important processes that can affect life loss, while depending only on readily available data sources and requiring only a reasonable level of effort to implement. It consists of the following internal modules:

1) Loss of Shelter – including prediction of building performance

2) Warning and Evacuation – including a transportation model, which accounts for traffic congestion and the effects of flooding in blocking road segments and on vehicle instability

3) Loss of Life – which is based on scale-independent empirical relationships developed by McClelland and Bowles (2000)

Estimated flooding conditions are obtained from an external dam break flood routing model. LIFESim can be run in deterministic or uncertainty modes. The uncertainty mode provides estimates of life loss and other variables relating to warning and evacuation effectiveness as probability distributions.

HEC-FIA

HEC-FIA, which is provided by USACE's Hydrologic Engineering Center, is based on a simplified LIFESim approach. It is a GIS-enabled model for estimating flood impacts due to a specific flood event. The software tool can generate required economic and population data for a study area from readily available data sets and use those data to compute urban and agricultural flood damage, area inundated, number of structures inundated, population at risk and loss of life. All damage assessments in HEC-FIA are computed on a structure-by-structure basis using inundated area depth grids. It assumes a uniform evacuation speed perpendicular to the inundation boundary. Peak inundation depth is considered but not the effects of flow velocity on the stability of people, structures and vehicles.

20.3.3 Sources of further guidance relevant to valuation and assessment

Table 20.12 presents potential methods and identifies the key information sources for each of the receptors. The method used may change dependent upon the inundation mapping category – total destruction, partial destruction or flood inundation.

Consideration should also be given as to whether national averages are used or whether correction is made for regional differences in contents and rebuilding costs. It is rarely necessary to evaluate most of the items in the list and it should be assessed as to which could make a difference to the results (say >10% change in outcome) and then limit the analysis to these major items.

Table 20.12 Outline methodology and datasets for assessment of losses from dam break

Potential receptor	Potential methods	Information
Residential properties	As this is an on–off event (compared with the cumulative events considered in fluvial and coastal flood risk management), total destruction will be demolition and full rebuilding costs, plus full contents value, plus cost of replacement housing while the property is rebuilt. Current average valuations for the subject area can be obtained from the Land Registry.	FHRC Land Registry National Property Database (NPD)/ Address-point
	Flood inundation damages can be calculated using FHRC data as an average damage per property. Although a range of depths and flood durations could be considered, for dam break it is normally more	

Potential receptor	Potential methods	Information
	appropriate to adopt a single average value. Consideration for flooding warning can be an optional consideration.	
Non-residential properties	Total and partial destruction are likely to result in write-off of properties based on the consideration above. Valuation can be based on the local statistical m^2 average taken from the 2001 Census information. Floor areas and property type may be available in National Property Database (NPD) if the Environment Agency provide this. However, if this information cannot be gathered from the Environment Agency then MasterMap information could be manipulated to gather these data.	FHRC UK Statistics MasterMap NPD/Address -point
	Flood inundation damages can be calculated using FHRC data; an average damage per property can be calculated. A range of depths and flood durations can be considered in the pilot study to determine the most appropriate. Consideration for flooding warning can be an optional consideration. This may depend on the cause of failure.	
Susceptible properties such as hospitals and old people's homes	The methodologies outlined above for non-residential properties can be applicable for direct damage.	See above
	The high vulnerability of the patrons is unlikely to be required in the damages evaluation however it may be relevant to fatality rate and thus likely loss of life (LLOL).	
Key service properties such as police stations and fire stations	The methodologies outlined above for non – residential properties can be applicable for direct damage.	See above
	The loss of emergency service providers during an event may not be required in the damages evaluation. However it might prove an important differentiator for prioritisation.	
Undertaker asset	Total destruction scenario – **u**ndertaker assets can be identifiable from data provided. Valuation information should be available from asset management/capital works team. Total destruction can assume cost of asset replacement. Valuation may also include the cost for loss of asset for duration of works.	Undertaker asset database/ valuation of assets/capital works cost database
	Partial destruction – cost of repairs can be available for a range of asset type. Valuation may also include the cost for loss of asset for duration of works.	
	Flood inundation valuation may include the cost for loss of asset for duration of works.	

Potential receptor	Potential methods	Information
Other key utilities/ critical infrastructure	The 2007 report by the House of Commons Committee of Public Accounts (NAO 2007) emphasised the importance of critical infrastructure, reinforcing the earlier Civil Contingencies Act (2004) which put in place regional and local resilience forums and legislation regarding economic key point (EKP). This may need to be bespoke and could consider some or all of write-off, consequential damage, the loss of service, the impact of service transfer and the cost of asset replacement.	OS maps Databases available to Category 2 responder
Reconstruction/ decommission	Single figure for asset. Can vary on asset size.	
Impact on business	Local business can only have a localised impact whereby they can incur a loss of business while shut and incur cost for staff and re-launch. This can be calculated using a 13% factor outlined in the Office for National Statistics' Blue Book. Although an Environment Agency approach would discount loss of trade as this is likely to be transferred, the approach would be different for financial loss.	Blue Book
	A manufacturing property may have a national importance for the UK economy and as such a bespoke assessment may be required	
Emergency service	FHRC guidance (following the 2007 floods Belwin claims) identified that 5.57% of the damage in that event could be attributed to emergency service response. Therefore this factor can be applied to the direct damages.	FHRC
Local utilities	FHRC guidance can be followed, although for local assets such as sub-stations this usually advises against including. Consequential damages are often reduced as transference of service can usually be applied.	FHRC
Undertaker production	Undertaker to provide historical information based on loss of previous assets. Standard rate can be built up for asset type or standard factor applied to all assessments.	Undertaker
Transport	FHRC guidance provides standard methodologies. Direct impacts relate only to the additional fuel consumption from delayed routes and standard compensation values for delayed train routes. Indirect impacts can be valued if required including the loss to business and the regional economy; this can depend on the scale of damage and disruption.	FHRC Local authority info Network Rail info
Development Areas	Can be valued based on predicted use or value of land for development.	Local authority

Potential receptor	Potential methods	Information
Health	Defra research demonstrated that the effects of stress and illness following a flood are difficult to estimate and based on a willingness to pay method. As this probability of flooding is very low the statistical damage is likely to be ~£6 per property.	FHRC
Environmental	Environmental assets have a value that society places on them. Value for all assets are available in the 2007 report by EFTEC (Economics for the Environment Consultancy); all datasets required are shown on the Magic website (www.magic.gov.uk) and available elsewhere (see Table 20.5).	EFTEC (2007)
Ecological	Ecological assets have a value that society places on them. Values for all assets are available in the EFTEC 2007 report; all datasets required are available in the Magic website.	Magic EFTEC (2007)
Historical	Historical assets have a value that society places on them. These can be difficult to value as many (such as war memorials) can have a special significance.	
Loss of service	Indirect damage can be difficult to value, as no standard methods are available. May need to be considered on a site-by-site basis.	
Motor vehicles	The review into the damages resulting from the 2007 flood showed that 3% (£80 million) were from written off vehicles. Although no standard method exists, recent research shows the average flood damage value (from write-off) as £5,000k; therefore only a vehicle count would be required (standard average per property could be used).	Property counts
Recreation	If the reservoir is used for recreational purposes this may be a loss of income that can be accounted for. This should disallow the transfer to other sites. The assessment could include the amenity value to the local community considering a willingness to pay method; standard examples may be available in the Yellow Manual (Penning-Rowsell et al. 1992).	Yellow Manual
Agriculture	Damage will be applicable to arable land as a one-off event only. No account of pastoral damages is normally included. Damage per hectare value available in the MCM is used.	Agricultural Land Classification FHRC
Fines, claims and legal	Data from other relevant or industry standard values. Could be applied across various categories.	
Temporary accommodation	Temporary accommodation can be required for a standard amount of time depending on the severity of the event. A monthly rate can be applied to durations up to one year.	Results of other analysis

For further summaries on data requirements, availability and quality see Table 2.3 in Defra and Environment Agency (2010).

20.4 Data issues

The availability and quality of data will have a major impact on the quality and level of detail of the assessment of damages and life loss, as well as programme (as the time required for data acquisition can be significant and needs to be included in any project programme).

An important consideration is security of data, as drawings and other data on dam construction could potentially assist terrorists in planning attacks on vulnerable installations. Defra has issued a security protocol on the use and dissemination of inundation maps. The user should have a planned and documented process to manage the security of documents relating to dams, agreed with the owner/undertaker of the dam.

OS MasterMap is produced by the Ordnance Survey and is a continually updated database that contains a variety of information structured into different product layers. These consist of:

- Topography layer – includes half a billion features on landscape representing features such as buildings, fields, fences, water bodies and intangible objects such as county boundaries
- Integrated Transport Network™ (ITN) layer – includes 5,445,000km of Great Britain's road network from motorways to local streets
- Address layer 2 – includes over 28 million addresses with classifications, unique property identifiers such as building name aliases, geographical addresses, objects without postal addresses such as churches and multiple occupancy information for flats
- Imagery layer – Seamless picture of Great Britain – between them they contain over 450 million geographical features found in the real world from individual addresses to roads and buildings. Every feature within the OS MasterMap database has a unique common reference (a TOID®) which enables the layers to be used together.

These data can be used to locate all water bodies within Great Britain using the topography layer. These can be verified using the imagery layer (Defra and Environment Agency 2010).

Although geographically based databases are invaluable in speeding up the process, they need to be used with care and reviewed critically. Examples of some of the issues are given below.

- Some datasets list the National Grid coordinates of the locations to which mail with valid addresses will be delivered. For residential properties, this is the physical location of the property. However, this may not be the case with commercial property where the delivery location may be a Post Office or a particular building receiving all the mail for a large organisation, and thus not the geographically correct location of the building vulnerable to flood risk. Commercial addresses can usually be identified, however, as an 'Organisation Name' is specified in the data.

- The lack of relative elevation data for residential property – for example, it can be impossible to distinguish between a block of flats and a development of sheltered accommodation which may both appear as many addresses with identical locations.
- Transient land uses commonly found in flood plain areas such as camp sites, waterside and water contact activities, fair grounds and so on may not be readily identifiable without further investigation.
- Some datasets identifies commercial properties which have been valued for business rates, but properties with zero business rates (for example, churches) may be omitted in this process.

Any automated assessments should be supported by visual checks, both on the ground and on maps, since there is a risk in some areas that significant numbers of properties may fall partially within a flood envelope, such as where gardens run down from houses towards streams running behind them. Consideration of the sensitivity of modelling results to the accuracy of the ground elevation data will identify many of these properties but will not identify the context, which could be significant when planning for dam break floods.

For buildings higher than one storey it is recommended that the number of storeys likely to be affected is included in calculating base population at risk (PAR), but the upper storeys are only included where the building was subject to total or partial structural destruction, or where the inundation depth reached these upper floors. The number of storeys may be entered as a multiplier on the building footprint and is therefore not necessarily an integer, where second (and higher) floors do not cover the whole building footprint. Where multiple storeys are entered it may be appropriate to adjust the occupant area and/or occupancy factor to provide an overall value applicable to all affected floors.

For residential property the assessment may generally be limited to the number of properties, with no consideration of different floor levels within one property, as PAR and property value relate to number of properties rather than floor area.

20.5 Further guidance relevant to the analysis and valuation of flood impacts

Table 20.13 Guidance and R&D documents relevant to assessment

Relevance	Document
Standards for damages valuation	Green Book (HM Treasury 2003)
Principals of flood damage	*Flood and Coastal Erosion Risk Management Appraisal Guidance* (Environment Agency 2010b)
Methods and data sets for flood damages (properties, transport, emergency service and so on)	*The Benefits of Flood and Coastal Risk Management: A Manual of Assessment Techniques* (FHRC 2010)
Relevant recent damages records	*The Costs of the Summer 2007 Floods in England* (Environment Agency 2010c)

Relevance	Document
Risk to life (fluvial)	*Assessing and Valuing the Risk to Life from Flooding* (Defra 2008a)
Life valuation	*The Accidents Sub Objective Transport Analysis Guidance Unit 3.4.1* (Department for Transport 2009)
Agriculture	*Supplementary Note to Operating Authorities Valuation of Agricultural Land And Output for Appraisal Purposes* (Defra 2008b)
Human health	*The Appraisal of Human Related Intangible Impacts of Flooding* (Defra 2005)
Environmental/ecological	*Flood and Coastal Erosion Risk Management: Economic Valuation of Environmental Effects* (EFTEC 2007)
Recreation	*The Economics of Coastal Management: A Manual of Benefit Assessment Techniques* (Penning-Rowsell et al. 1992)
Business impacts	*The Benefits of Flood Alleviation: A Manual of Assessment Techniques* (Penning-Rowsell and Chatterton 1977)

21 Guidance on risk analysis and evaluation

21.1 Introduction

There is now a wide range of literature on risk analysis and evaluation, as summarised in the boxes within this section. This section of the guide is therefore limited to material which is necessary to explain the basis of the guidance in Part 1, or to signpost to more detailed texts, or may be helpful in interpreting and applying Part 1.

This section is structured by a sub-section on strategic issues, followed by the steps on risk analysis, namely Steps 1c, 2e and 3.

Box 21.1 Key references for risk analysis and evaluation in relation to dam safety

General

- BS ISO 31000:2009 Risk management – principles and guidelines
- BS EN 31010:2010 Risk management techniques
- BS 31100:2011 Risk management – code of practice and guidance for the implementation of BS ISO 31000
- BS EN 60812:2006 Analyses techniques for system reliability – procedure for failure mode and effects analysis (FMEA)
- *The Orange Book. Management of Risk – Principles and Concepts* (HM Treasury 2004)

Dam safety

- *Dam Safety Risk Analysis. Best Practices Training Manual* (Reclamation 2009-2011) – 34 chapters on all aspects of risk analysis
- *Risk and Uncertainty in Dam Safety* (Hartford and Baecher 2004)
- ICOLD Bulletin 130 Risk Assessment in Dam Safety Management (ICOLD 2005)

Fluvial and coastal flood defences

- *Management of Flood Embankments – A Good Practice Review.* FD2411/TR1 (Defra and Environment Agency 2007a)
- *Risk, Performance and Uncertainty in Flood and Coastal Defence – A Review.* FD2302/TR1 (Defra and Environment Agency 2002)
- *Performance and Reliability of Flood and Coastal Defences*. FD2318/TR1 and FD2318/TR2 (Defra and Environment Agency 2007b)

(Joint Defra/EA Flood and Coastal Erosion Risk Management R&D Programme)

21.2 Key issues and concepts

21.2.1 Alternative criteria to evaluate risk

A high level comparison of different criteria to evaluate risk is summarised in Table 21.1. This guide is based on the HSE framework for tolerability of risk.

Table 21.1 Alternative groups of risk criteria

Risk criteria	Principle
Equity based	'All individuals have unconditional rights to certain levels of protection'
Utility based	'Comparison between the incremental benefits of the measure to prevent the risk of injury or detriment, and the cost[1] of the measure'
Technology based	'The idea that a satisfactory level of risk prevention is attained when 'state of the art' control measures (technological, managerial, and organisational) are employed to control risks whatever the circumstances.'
Applied	Typically a hybrid of the pure criteria summarised above. The generalised framework for tolerability of risk (TOR) developed by HSE (2001) is intended 'to capitalise on the advantages of each of the above 'pure criteria' while avoiding their disadvantages' and to resemble the decision process that people use in 'everyday life'.

Notes: Source: HSE (2001, paragraph 119)

[1] Where cost is considered in broad terms, which may include time and effort in addition to monetary aspects.

There is currently no defined standard for what constitutes a tolerable level for risk posed by a dam. This guide is therefore based on what is understood to be current good practice in other industries, as defined in *Reducing Risk Protecting People* ((R2P2), published by HSE (2001) and summarised in section 21.2.6.

However, some important aspects of regulation of risk by HSE in other industries are set out below to inform the assessment of risk from reservoirs.

First, an important and not always well-recognised aspect of the regulation of workplace risk in the UK is that, although the Health and Safety Commission (HSC) approves a safety case under the Control of Major Accident Hazards Regulations (COMAH), it is constituted in such a way that it the courts have so far chosen not to challenge that decision (Le Guen 2010). This appears to provide a high level of protection against legal liability for the hazard owner. In fact in this situation there is considered to be a co-responsibility between the owner for the hazard and HSE as the regulator that has approved the safety case. Clearly it is a desirable situation for the hazard owner. However, this is a situation that does not exist under the arrangements for regulating reservoir safety in the UK under the 1974 and 2010 acts.

Secondly, another feature of regulation by HSE, which has particular legal significance for the duty holders, is its Approved Codes of Practice (ACOPs). According to Le Guen (2010) these ACOPs:

> 'clarify particular aspects of the general duties and regulations, and are HSC's way of spelling out their implications. ACOPs have a special guidance status. If employers are prosecuted for a breach of health and

safety law, and it is proved that they have not followed the relevant provisions of the Approved Code of Practice, a court can find them at fault unless they can show that they have complied with the law in some other way. Accordingly, the HSE agreed in 1996, following consultation, that it would limit the use of guidance having the status of an ACOP to cases where five [four] conditions were met. These are when:

- there is clear evidence of a significant or widespread problem;
- the overall approach being taken to an area of risk is by amplifying general duties in the HSW Act or preparing goal-setting regulations (see paragraph 4);
- there is a strong presumption in favour of a particular method or particular methods that can be amplified in an ACOP in support of the general duties or goal setting regulations to give authoritative practical guidance;
- the alternative is likely to be more prescriptive regulation;
- guidance, which is not law but gives advice on measures available and what is good practice.

It is presumed that the various engineering guides relating to reservoir safety are not Approved Codes of Practice.

HSE (2001) also states that risk in the broadly acceptable region 'would not usually require further action to reduce risks unless reasonably practicable measures are available.' This statement is interpreted that the ALARP principle still applies in the broadly acceptable region. For this reason, and because the level of assurance of legal defensibility that is provided through regulation by HSE as described above is not available in other countries, the broadly acceptable region has not been separately defined in other project-specific tolerable risk frameworks such as ANCOLD (2003), New South Wales Dams Safety Committee (2006) and USACE (2010). The exception is reservoirs that pose a risk that is low enough to meet the tolerable risk requirements, which are excluded from regulation.

Lastly, standards may change with time. Attention is drawn to the following references which chart on-going consultation and discussion on refining standards:

- consultation document (CD212) published by HSE on behalf of the Cross-government Task Group on Societal Risk (HSE 2007)
- HSE Research Report RR703, *Societal Risk: Initial briefing to Societal Risk Technical Advisory Group* (HSE 2009)

21.2.2 Scoping risk analysis (Step 1c)

This is a critical stage in defining both expectation for the output and the level of effort required to achieve the defined output.

Reference should be made at this stage to the objectives of the risk assessment (see Chapter 2) as these will have a bearing of the 'scenario construction'; for example, if the objective is to consider a 'worst case' scenario then it would not be prudent to consider the impact on people in the downstream area at a time when the majority of local residents would not be in the area (that is, away at work during a weekday).

Similarly one would not choose a 'sunny day' condition of the reservoir (that is, not in flood) unless there was a specific reason for doing so.

Similarly the emphasis can vary with examples being:

- on the range of consequences, when the output is presented as total probability of failure with the effect on the different receptors separated out
- on the probability of individual modes of failure, with consequences simplified to average societal life loss and individual risk

Caution: The risk scoping stage defines the number of variables that will form the output from the risk assessment, with suggested levels for Tiers 1 and 2 shown in Tables 3.1 and 7.5. Plotting large numbers of failure modes each with their own separate consequences will produce a large number of variables, which should only be handled through bespoke software.

21.2.3 Dependence and independence of failure modes

Failure modes may be dependent or independent of one another; where this is neglected the overall probability of failure would be too high (for example, >100%). As probabilities of dam failure are normally modest this effect is only important where the highest probability of failure is greater than say 1 in 5 (20%) chance per year.

i. In the case of **full dependence,** if one failure mode occurs it can be assumed that all other failure modes will have occurred and hence the associated likelihood of occurrence is simply the maximum value, and can be determined simply by:

$$P_f = \max\lfloor P_{fi}, P_{fj}, P_{fc} \rfloor \qquad \text{Eq. 21.1}$$

where P_f is the overall probability of failure and P_{fi} is the probability of a single failure mode.

ii. In the case of **independence,** any chance of any given failure occurring is not influenced by other failure modes. The overall chance of failure can be given through an application of de Morgan's law to give:

$$P_{fc} = 1 - (1 - P_{fi}).(1 - P_{fj}) \qquad \text{Eq. 21.2}$$

where P_f is the overall probability of failure and P_{fi} is the probability of a single failure mode.

The two values determined through assumptions of independence and dependence provide bounds on the overall probability of failure (Sayers et al. 2001). These upper and lower bounds can be multiplied with the upper and lower bounds of measures of potential consequences to provide upper and lower bounds of annual expected risk. A mean value can be determined from these.

21.2.4 Uncertainty and sensitivity analysis

Uncertainty and sensitivity analysis are closely related but are not the same. Uncertainty seeks to enable the decision-maker to better understand the confidence within the evidence presented and the choices taken. Sensitivity analysis seeks to highlight to the decision-maker those aspects of the analysis which are most sensitive to the evidence presented and the choices made.

Both types of analysis can add value to the risk analysis process by, for example, highlighting those areas of the inundation zone where there is more or less confidence in the assessment of flood risk.

By maintaining uncertainty as a single layer within the risk analysis (rather than multiple layers within the supporting consequence and hazard assessment), the user is able to choose the complexity or simplicity of the uncertainty analysis. The user may even choose not to record or present uncertainty (as it doesn't affect the risk per se), although it can be applied to any of the tiers as the users feels appropriate. However, the strong recommendation here is to be explicit about the uncertainty – even where it is difficult to estimate, the engineer undertaking the analysis is the best able to provide this estimate. Table 21.2 summarises the different approaches to assessing the impact of uncertainty.

Table 21.2 Different approaches to assess the impact of uncertainty

Type of modelling	Description	Comment
Sensitivity testing	Examining a number of scenarios without attaching probabilities to them. Enables preliminary exploration of the potential consequences of uncertainty in future performance and on decision-making.	Sensitivity testing can be used to identify by how much key variables can change before a different preferred option is identified. Sensitivity testing usually involves varying each parameter in turn with other parameters held at their 'best estimate' value. It is often appropriate to conduct some sensitivity tests before embarking on more thorough simulation methods. Common place in various aspects of engineering, including geotechnical analysis associated with exploration of slope stability (for example) where soil properties are varied through a range of values to ensure that the factor of safety is robust to plausible values.
Simulation	Representing variables and model parameters as probability distributions and propagating these through the risk model using a numerical sampling method. Correlation between variables and parameters can be included simply where known or estimated where necessary to ensure neutral or slightly conservative estimates. The uncertainty in the output risk is then established through the analysis of multiple samples	Can be computationally and data intensive. However, when coupled with expert judgment to identify the most important parameters and associated distributions, much more simple applications can be developed. For example, if a parameter has a narrow confidence interval (small uncertainty) and has a minor effect on the response, then it is feasible to consider it as known (that is, no uncertainty). However, the analysis might become more complicated if it is necessary to consider the different sources of uncertainty as separate elements, and structure the analysis to calculate specific uncertainty sources before combining these analyses in an overall simulation.

Box 21.2 Procedure for tackling uncertainty in R2P2 (HSE 2001)

The suggested procedure for dealing with uncertainty given in R2P2 is shown in Appendix 1 paragraph 10 of that publication. It includes where consequences are uncertain 'consider putative consequences' and where likelihood is uncertain 'emphasis on consequences for example if serious/ irreversible or need to address social concerns'.

21.2.6 Criteria to evaluate tolerability of risk

Box 21.3 Key references on tolerability of risk

- *Reducing Risk, Protecting People* (HSE 2001)
- HSE's ALARP Suite of Guidance (HSE 2001-2006) – series of six notes
- *PADHI (Planning Advice for Developments near Hazardous Installations)* (HSE 2011)

A key outcome from any risk assessment is whether the dam is safe enough, or whether measures are required to reduce the risk (which in principle could, as a last resort, for very high risk dams comprise removal of the dam). The subject of what constitutes tolerable risk is a complex subject, with many publications discussing the issues and what is considered tolerable varying between industries and countries. In UK these have varied over time, from the initial guidelines for nuclear power plants (HM Nuclear Installations Inspectorate 1979) to HSE (2001). Papers discussing criteria in UK for dams include Hughes and Gardener (2004) and Brown et al. (2012).

One of the issues in evaluating risk is the relative importance given to individual risk compared to cumulative impact, for example, is 1m deep flooding of 50 houses higher impact than death of two people in an isolated house? Current criteria for both societal and individual risk as set out in R2P2 are summarised in Table 21.3; it is suggested that these are applied to reservoirs in UK.

Table 21.3 Criteria for tolerability of risk to human life in UK suggested in R2P2 (HSE 2001)

Type of risk (definition in Glossary)	Boundary suggested in R2P2 (paragraph number in R2P2)	
	Tolerable and unacceptable	Tolerable and the broadly acceptable
Individual risk	For members of the public who have a risk imposed on them 'in the wider interest of society' this limit is judged to be 1 in 10,000 (10^{-4}) (132)	Individual risk of death of one in a million per annum (10^{-6}) (130)
Societal risk	The risk of an accident causing the death of 50 people or more in a single event should be regarded as intolerable if the frequency is estimated to be more than one in five thousand per annum'. (136)	No specific advice. Older publications suggest that it may be two orders of magnitude lower than the boundary for broadly acceptable.

It is common in risk management to refer to safety goals as risks reduced to 'as low as reasonably practicable'. This is widely known as the ALARP principle and applies in the tolerable zone of Table 21.3. To implement the ALARP principle a 'gross disproportion' test needs to be applied to determine the balance between individual risks and societal concerns, including societal risks. The gross disproportion is between the cost of an additional risk reduction measure and the estimated amount of that reduction in risk. There are various ways of doing this.

Application of this principle and estimation of the proportion factors is presented in the guidance on Step 3c in Tier 2.

Levels of tolerability for potential impact on other receptors such as cultural and environmentally important sites have not been defined, and the user should refer to UK government and European guidance current at the time of the assessment.

21.2.7 Decision-making (Step 3e)

Chapter 2 of this guide describes how the objectives and contact of the risk assessment should be established before starting any analysis. It is these that define how decisions are made as to the tolerability of existing risk, or whether risk reduction measures are required.

As well as the estimation of risk, as well as a realistic assessment of the uncertainty of the estimate it is necessary to consider other factors as described in Step 3d of each tier, such that both are used in the decision-making process.

Decision-making is often by a company director or other manager who is non-specialist in risk assessment. The engineer carrying out the assessment should therefore ensure that the risk assessment process and outcomes are presented in a way that is intelligible by non-technical people.

21.3 Basis of tiered methodology for risk analysis and evaluation

Escalation of analysis between tiers is shown in Table 21.4. The process is similar between the tiers, so it is level of detail of presentation.

Table 21.4 Comparison of detail of risk analysis and evaluation with tier

	Tier 1	Tier 2	Tier 3
Risk analysis – social	Qualitative description of position of plot on matrix of six steps of probability (extreme to very low) vs. five levels of consequences	F-N plot	Bespoke depending on client objectives. Output may include how level of risk varies with reservoir load, and so on.
Individual	Not included	Product of individual vulnerability and APF	
Economic		£/year product of consequences and APF	
Other		Description of likelihood and level of	

	Tier 1	Tier 2	Tier 3
		impact	
Risk evaluation			
Tolerability			
Societal	6 × 5 matrix based on F-N plot	Numeric; apply R2P2 criteria	
Individual	Not evaluated		
Proportionality of options to reduce risk	Simple quantitative comparison of whole life costs to benefits	ALARP calculation	

21.4 Supporting information to risk analysis and evaluation

21.4.1 Option assessment (Step 3b) – identification and costing

Assessment of risk reduction options should start at a simple level of considering the whole range of groups of options (see Table 6.2 in Tier 1), as it is often self-evident that a simple non-structural measure would significantly reduce the risk. Where appropriate this can develop into more formal consideration of structural options, often in the form of a pre-feasibility or feasibility report embodied in a formal ALARP analysis to define the appropriate level of risk reduction.

Where a formal ALARP analysis is appropriate, this would normally include a risk assessment of the current condition of the dam, being repeated with the dam in the anticipated condition after the works.

Appendix 2 to R2P2 (HSE 2001) provides a useful commentary on issues relating to assessing risk reduction options and reference should be made to that publication when assessing risk reduction options.

References

Aboeleta, M.A. and Bowles, D.S.	2005	*LIFESim: A Model for Estimating Dam Failure Life Loss.* Draft report to US Army Corps of Engineers and ANCOLD. Institute for Dam Safety Risk Management and Utah State University, Logan, UT.
Allsop, N.W.H., McKenna, J.E., Vicinanza, D. and Whittaker, T.J.T.	1996a	*New design formulae for wave loadings on vertical breakwaters and seawalls*. 25th International Conference on Coastal Engineering, September 1996, Orlando, FL. ASCE, New York.
Allsop, N.W.H. and Vicinanza, D.	1996	*Wave impact loadings on vertical breakwaters: development of new prediction formulae*. In Proceedings 11th International Harbour Congress, Antwerp, Belgium.
Allsop, N.W.H., Vicinanza, D. and McKenna, J.E.	1996b	*Wave forces on vertical and composite breakwaters*. Research Report SR 443. HR Wallingford, Wallingford.
Allsop, N.W.H., Kortenhaus, A. McConnell, K.J. and Oumeraci, H.	1999	*New design methods for wave loadings on vertical breakwaters under pulsating and impact conditions*. In Proceedings Coastal Structures'99, Santander, Spain. Balkema, Rotterdam.
Allsop, N.W.H.	2000	'Wave forces on vertical and composite walls', in *Handbook of Coastal Engineering*, ed. J. Herbich, pp. 4.1–4.47. McGraw-Hill, New York.
Allsop, N.W.H.	2010	*Waves and wave overtopping on reservoir structures*, Proc ICE, Dams & Reservoirs, 20, (2), 59-68, Thomas Telford.
Allsop, W., Kortenhaus, A. and Morris, M.	2007	*Failure Mechanisms for Flood Defence Structures.* Report T04-06-01, FLOODsite. Available at: http://www.floodsite.net [Accessed February 2013].
ANCOLD	2003	Guidelines on Risk Assessment. Australian National Committee on Large Dams (ANCOLD), Sydney.
ANCOLD	2011	*Guidelines on the Consequence Categories for Dams.* April draft for consultation. Australian National Committee on Large Dams (ANCOLD), Sydney.
ASDSO	1988	*Training Aids for Dam Safety. Module: Evaluation of Concrete Dam Stability.* Interagency Committee on Dam Safety, US Association of Dam Safety Officials, Denver, CO.
Battjes, J.A.	1974	*Surf similarity*, Proceedings 14th Coastal Engineering Conference, Copenhagen, pp.466–479. ASCE, New York.
Binnie, G.M.	1981	*Early Victorian Water Engineers.* Thomas Telford, London.
Binnie & Partners	1991	*Estimation of flood damage following potential dam*

		failure: guidelines. *1989 Report for DOE*. FR/D 0003. Foundation for Water Research, Marlow.
Bishop, A.W. and Morgenstern, N.	1960	Stability coefficients for earth slopes. *Géotechnique,* 10(4), 129-153.
Blackmore, P.A. and Hewson, P.	1984	Experiments on full scale impact pressures. *Coastal Engineering*, 8, 331-346.
Bowles, D.S.	2004	*ALARP Evaluation: Using cost effectiveness and disproportionality to justify risk reduction.* Australian National Committee on Large Dams Conference, ANCOLD Bulletin 127:89106.
Brown, A.J. and Tedd, P.	2003	The annual probability of a dam safety incident at an embankment dam, based on historical data. *International Journal of Hydropower & Dams,* 10(2), 122-126.
Brown, A.J. and Gosden, J.D	2004	*An Interim Guide to Quantitative Risk Assessment for UK Reservoirs*. Thomas Telford, London.
Brown, A.J., Chapman, A, Oden, J.D. and Smith, J.J.B.	2008	*The influence of infrastructure embankments on the consequences of dam failure*. Proceedings British Dam Society 15th Biennial Conference, Warwick, 10–13 September 2008, Paper 14, pp. 80-94.
Brown, A.J. and Peters, A.	2007	Update on the use of the Interim Guide to Quantitative Risk Assessment. *Dams and Reservoirs*, 17(1), 19-24.
Brown, A.J., Yarwood, G., King, S.J. and Gosden, J.D.	2008	*Application of the Interim Guide to Quantitative Risk Assessment across multiple dam owners by multiple Jacobs offices*, Proceedings British Dam Society 15th Biennial Conference, Warwick, 10–13 September 2008, Paper 13, pp. 65-79.
Brown, A.J., Gosden, J.D. and Gotch, P.	2012	*Observations on the boundary between high and lower risk reservoirs*. Proceedings British Dam Society 17th Biennial Conference, Leeds, 12–15 September 2012, Paper 3.2.
Brown & Root	2002	*Reservoir safety – Floods and Reservoir Safety Integration*. Research Report for Defra. Project WT01005.
BSI	1995	BS 6399 Part 2. Loadings for buildings: Code of practice for wind loads. British Standards Institution, London.
BSI	2000	BS EN 12056-3:2000 Gravity drainage systems inside buildings. Roof drainage, layout and calculation. British Standards Institution, London.
BSI	2006	BS EN 60812:2006. Analysis techniques for system reliability. Procedure for failure mode and effects analysis (FMEA). British Standards Institution, London.
BSI	2007	BS EN 61025:2007. Fault tree analysis (FTA). British Standards Institution, London.

BSI	2008	BS ISO 3846:2008. Hydrometry. Open channel flow measurements using rectangular broad crested weirs. British Standards Institution, London.
BSI	2009	BS 6031:2009. Code of practice for earthworks. British Standards Institution, London.
BSI	2009	BS ISO 31000:2009. Risk management. Principles and guidelines. British Standards Institution, London.
BSI	2010	BS EN 31010:2010. Risk management. Risk assessment techniques. British Standards Institution, London.
BSI	2011	BS 31100:2011 Risk management. Code of Practice and guidance for the implementation of BS ISO 31000. British Standards Institution, London.
BSI	2011	BS EN 62502:2011. Analysis techniques for dependability. Event tree analysis (ETA). British Standards Institution, London.
Burnett, J.B.	1992	A regional engineering geological study of the London Clay in the London and Hampshire Basins. *Quarterly Journal of Engineering Geology and Hydrogeology*, 7, 257-295.
Bussell, M.N.	1995	*Design and Construction of Joints in Concrete Structures*. Report 146. CIRIA, London.
Camfield, F.E.	1991	Wave forces on wall, *Journal of Waterway, Port, Coastal, & Ocean Engineering*, 117(1), 76-79.
Carter, D.A.	1995	'The Scaled Risk Integral – A Simple Numerical Representation of Case Societal Risk for Land Use Planning in the Vicinity of Major Accident Hazards', *Loss Prevention and Safety Promotion in the Process Industries*, Volume II, Elsevier, 1995.
CEH	1999	*Flood Estimation Handbook*. Five volumes. Centre for Ecology & Hydrology, Wallingford.
Chandler, R.J.	1969	The effect of weathering on the shear strength properties of Keuper marl. *Geotechnique*, 19, 321-334.
Chandler, R.J.	1972	Lias Clay: weathering processes and their effect on shear strength. *Geotechnique*, 22, 403-431.
Chandler, R.J. and Apted, J.P.	1988	The effect of weathering on the strength of London clay. *Quarterly Journal of Engineering Geology and Hydrogeology*, 21, 59-68.
Charles, J.A., Abbis C.P., Goschalk, E.M, and Hinks, J.L.	1991	*An Engineering Guide to Seismic Risk to Dams in the UK*. BRE Report 210. BRE, Watford.
Charles, A., Tedd, P., Hughes, A.K. and Lowenbury, H.T.	1996	*Investigating Embankment Dams. A Guide to the Identification and Repair for Defects*. BRE Report. BRE, Watford.

Chow, V. T.,	1986	Open Channel Hydraulics. McGraw-Hill, New York.
Chrimes, M.	2009a	Ahead of the game – masonry dam design in the British colonies 1800–1900, part 1: 1800–1872. *Dams & Reservoirs,* 19(2), 55-66.
Chrimes, M.	2009b	Ahead of the game – masonry dam design in the British colonies 1800–1900, part 2: 1872–1900. *Dams & Reservoirs,* 19(4), 171-183.
CIBSE	2003	*Fire Engineering*. 2nd edition. Chartered Institution of Building Services Engineers Guide E.
CIRIA	2003	*Crane Stability on Site – An Introductory Guide*. 2nd edition. CIRIA Report C703. CIRIA, London. [Previously published in 1996, revised 200, as SP131]
CIRIA	2013	*International Levee Handbook.* Project coordinated by CIRIA, publication expected 2013; see www.leveehandbook.net.
CIRIA, CUR and CETMEF	2007	*The Rock Manual. The Use of Rock in Hydraulic Engineering*, 2nd edition.C683, CIRIA, London.
Conroy, P., De Rosa, S. and Yeowell, A.	2011	*Deterioration Rates of Long-Life, Low Probability of Failure Assets: Project Report*. UK Water Industry Research Ltd. Report ref 11/WM/13/2.
Cross, R.H.	1967	Tsunami Surge Forces, *Journal of the Waterways & Harbors Division, Proc. ASCE*, 93 (WW4), 201-231.
Cuomo, G., Allsop, N.W.H. and Takahashi, S.	2010a	Scaling wave impact pressures on vertical walls. *Coastal Engineering*, 57(6), 604-609.
Cuomo, G., Allsop, N.W.H., Bruce, T. and Pearson, J.	2010b	Breaking wave loads at vertical sea walls & breakwaters. *Coastal Engineering*, 57(4), 424-439.
Cuomo, G., Piscopia, R. and Allsop, N.W.H.	2011	Evaluation of wave impact loads on caisson breakwaters based on joint probability of impact maxima and rise times. *Coastal Engineering*, 58(1), 9-27.
Cyganiewicz, J.M., Engemoen, W.O. and Redlinger, C.G.	2007	Bureau of Reclamation experience with evaluating internal erosion of embankment dams. In: Fell, R. and Fry, J.-J., eds., *Internal Erosion and Piping of Dam and their Foundations,* Proceedings of Workshop, 25–27 April 2005, Aussois, France. Taylor & Francis, London.
de Vroeg, J. H., Kruse, G. A. M. & van Gent, M. R. A.	2002	*Processes relating to breaching of dikes.* DC030202/H3803. The Netherlands: Delft Cluster.
Defra	2003	*Flood and Coastal Defence Project Appraisal Guidance. FCDPAG3 Economic Appraisal. Supplementary Note to Operating Authorities. Revisions to economic appraisal procedures arising from the new HM Treasury 'Green Book'*. Defra, London.

Defra	2005	*The Appraisal of Human Related Intangible Impacts of Flooding*. Defra, London
Defra	2006	Supplement No 1 to Interim Guide. Published on Defra website with informal consultation on draft Guide to Emergency Planning. Defra, London.
Defra	2008	*Supplementary Note to Operating Authorities, Assessing and Valuing the Risk to Life from Flooding for Use in Appraisal of Risk Management Measures.* Defra, London.
Defra	2018	*Supplementary Note to Operating Authorities. Valuation of Agricultural Land And Output for Appraisal Purposes.* Defra, London.
Defra and Environment Agency	2002	*Risk, Performance and Uncertainty in Flood and Coastal Defence – A Review*. Defra/Environment Agency Flood and Coastal Defence R&D Programme. Report FD2302/TR1. Environment Agency, Bristol.
Defra and Environment Agency	2006	*Flood Risks to People*. TR1 Methodology TR2 Guidance. Defra/Environment Agency Flood and Coastal Defence R&D Programme. Report FD 2321. Environment Agency, Bristol.
Defra and Environment Agency	2007a	*Management of Flood Embankments – a Good Practice Review.* Joint Defra/Environment Agency Flood and Coastal Erosion Risk Management R&D Programme. FD2411/TR1. Environment Agency, Bristol.
Defra and Environment Agency	2007b	*Performance and Reliability of Flood and Coastal Defences*. Joint Defra/Environment Agency Flood and Coastal Erosion Risk Management R&D Programme. FD2318/TR1&2. Environment Agency, Bristol.
Defra and Environment Agency	2007c	*Reducing the Risk of Embankment Failure Under Extreme Conditions A Framework for Action*. Defra/Environment Agency Flood and Coastal Defence R&D Programme. Technical Report FD2411/TR2. Environment Agency, Bristol.
Defra and Environment Agency	2007d	*Soil Piping Tests on Thorngumbald Flood Embankment.* Defra/Environment Agency Flood and Coastal Defence R&D Programme. Report FD2411/TR3. Environment Agency, Bristol.
Defra and Environment Agency	2010	*Scoping the Risk Assessment Process for Small Reservoirs*. Joint Defra/Environment Agency Flood and Coastal Erosion Risk Management R&D Programme. Report FD2640/TR1. Environment Agency, Bristol.
Defra and Environment Agency	unpub	*Scoping the Process for Determining Acceptable Levels of Risk in Reservoir Design.* Joint Defra/Environment Agency Flood and Coastal Erosion Risk Management R&D Programme. Project FD2641. (*Completed 2010 but unpublished. Supported by three technical papers, of which Paper 2 (Task 3: Assessment of reservoir characteristics affecting reservoir safety risks) includes*

		anonymised summary plots of output from risk assessments of UK reservoirs. Copies available from Defra).
Department for Education and Skills	2004	*Briefing Framework for Secondary School Projects.* Department for Education, London.
Department for Education and Skills	2005	*Briefing Framework for Primary School Projects.* Department for Education, London.
Department for Transport	2007	*Valuation of the Benefits of Prevention of Road Accidents and Causalities*. DoT, London.
Department for Transport	2012	*The Accidents Sub Objective Transport Analysis Guidance Unit 3.4.1*. DoT, London. Available at: http://www.dft.gov.uk/webtag/documents/expert/unit3.4.1.php [Accessed March 2013]
Dolen, T.P.	2011	*Selecting strength input parameters for structural analysis of aging concrete dams*. 21st Century Dam Design – Advances and Adaptations, 31st Annual USSD Conference, San Diego, California, 11–15 April 2011. US Bureau of Reclamation, Denver, CO.
Douglas, K., Spannagle, M. and Fell, R.	1998	*Estimating the probability of failure of concrete and masonry gravity dams.* ANCOLD Bulletin. No. 112, 53-63.
Douglas, K., Spannagle, M., and Fell, R.	1999	Analysis of concrete and masonry dam incidents. *The International Journal on Hydropower & Dams*, 6(4),108-115.
Duncan and Wright	2005	Soil Strength and Slope Stability. John Wiley & Sons, Hoboken, N.J.
Eddleston, M. and Carter, I.C.	2006	Comparison of methods to determine the probability of failure due to internal erosion in embankment dams. In: *Improvements in Reservoir Construction, Operation and Maintenance*, Proceedings 14th Conference of British Dam Society, Durham, 6–9 September 2006, Paper 51. Thomas Telford, London.
Eddleston, M.	2012	*Exploring the potential use of a risk based approach to assessing the geotechnical well-being of the slopes of old embankment dams*. Proceedings British Dam Society 17th Biennial Conference, Leeds, 12–15 September 2012, Paper 3.5, pp. 206-218.
EFTEC	2007	*Flood and Coastal Erosion Risk Management: Economic Valuation of Environmental Effects. Handbook for the Environment Agency for England and Wales*. Economics for the Environment Consultancy (EFTEC), London.
Environment Agency	2009a	*Reservoir Inundation Mapping – Trial Study.* Environment Agency, Bristol.

Environment Agency	2009b	*Desktop Review of 2D Hydraulic Modelling Packages.* Report SC080035. Environment Agency, Bristol.
Environment Agency	2010a	*Guidance for the Design and Maintenance of Stepped Masonry Spillways*. Report SC080015. Environment Agency, Bristol.
Environment Agency	2010b	*Flood and Coastal Erosion Risk Management Appraisal Guidance. Supporting Document for the Appraisal Summary Table.* Environment Agency, Bristol.
Environment Agency	2010c	*The Costs of the Summer 2007 Floods in England.* Report SC070039/R1. Environment Agency, Bristol.
Environment Agency	2011a	*Lessons from Historical Dam Incidents*. Report SC080046/R1. Environment Agency, Bristol.
Environment Agency	2011b	*Modes of Dam Failure and Monitoring And Measuring Techniques.* Report SC080048/R1. Environment Agency, Bristol.
European Commission [Floods Directive]	2007	Directive 2007/60/EC of the European Parliament and of the Council of 23 October 2007 on the assessment and management of flood risks. *Official Journal of the European Union*, L288, 27-34.
European Commission [Water Framework Directive]	2000	Directive 2000/60/EC of the European Parliament and of the Council of 23 October 2000 establishing a framework for Community action in the field of water policy. *Official Journal of the European Communities*, L327, 1-73.
EurOtop	2007	*Wave overtopping of sea defences and related structures: Assessment manual.* UK Environment Agency, Coastal Engineering Research Council, Germany and Rijkswaterstaat, Netherlands. Available at: http://www.overtopping-manual.com/manual.html [Accessed February 2013]
Fell, R., Foster, M., Davidson, R., Cyganiewicz, J., Sills, G., Vroman, N. and Davidson, R.	2008a	*Risk Analysis for Dam Safety. A Unified Method for Estimating Probabilities of Failure of Embankment Dams by Internal Erosion and Piping, Draft guidance document dated August 21, 2008*. URS Australia Pty Ltd. Sydney, Australia [Prepared for the US Bureau of Reclamation and US Army Corps of Engineers].
Fell, R., Foster, M., Davidson, R., Cyganiewicz, J., Sills, G. and Vroman, N.	2008b	*Seepage and piping toolbox-initiation of internal erosion*. In: Proceedings 28th Annual USSD Conference, Portland, OR, 28 April to 2 May 2008.
Federal Energy Regulatory Commission (FERC)	2002	*Dam Safety Performance Monitoring Programme.* Chapter 14 of the Engineering Guidelines. FERC, Washington DC.
FHRC	2010	*The Benefits of Flood and Coastal Risk Management: A Handbook of Assessment Techniques. Multi-Coloured Handbook.* Flood Hazard Research Centre, Middlesex

		University, London.
FLOODsite	2007	European Research collaboration: 20 themes, each with executive summaries and multiple technical reports. Available at: http://www.floodsite.net [Accessed February 2013].
Foster, M., Fell, R., and Spannagle, M.	1998	*Risk assessment – estimating the probability of failure of embankment dams by piping.* ANCOLD 1998 Conference on dams.
Foster, M. and Fell, R.	2001	Assessing embankment dams that do not satisfy design criteria. *Journal of Geotechnology and Geoenvironmental Engineering*, 127(5), 398-407.
Froehlich, D.C.	1995	Peak outflow from breached embankment dam. *ASCE Journal of Water Resources Planning and Management* 121(1), 90-97.
Garner, S.J. and Fannin, R.J.	2010	Understanding internal erosion: a decade of research following a sinkhole event, *International Journal on Hydropower and Dams*, 17(3), 93-98.
Goda, Y.	1974	*New wave pressure formulae for composite breakwaters*. In: Proceedings14th International Conference on Coastal Engineering, pp. 1702–1720. ASCE, New York.
Goda, Y.	1985	*Random Seas and Maritime Structures.* University of Tokyo Press, Tokyo.
Gosden, J.D. and Dutton, D.	2006	Quantitative risk assessment in practice. In: *Improvements in Reservoir Construction, Operation and Maintenance*, Proceedings 14th Conference of British Dam Society, Durham, 6–9 September 2006, Paper 41. Thomas Telford, London.
Graham, W.	1999	*A Procedure for Estimating Loss of Life Caused by Dam Failure*. DSO-00-06. US Bureau of Reclamation, Denver, CO.
Hanson, G.J. and Simon, A.	2001	Erodibility of cohesive streambeds in the loess area of the Midwestern USA. *Hydrological Processes,* 20, 23-38.
Hanson, G.J. and Cook, K.R.	2004	Apparatus, test procedures and analytical methods to measure soil erodibility in situ. *Applied Engineering in agriculture,* 20(4), 455-462.
Hanson, G.J., Temple, D.M., Morris, M.W. and Hassan, M.A.A.M.	2005	*Simplified breach analysis model for homogeneous embankments: Part II, Parameter inputs and variable scale model comparisons*. In: 25th United States Society on Dams (USSD) Annual Conference, 6–10 June, 2005, Salt Lake City, UT.
Hanson, G.J. and Hunt, S.L.	2007	Lessons learned using laboratory jet testing method to measure soil erodibility of compacted soils. *Journal of Applied Engineering in Agriculture,* 23(3), 305-312.
Hanson, G.J., Tejral, R.D., Hunt, S L. and	2010	*Internal erosion and impact of erosion resistance*. In: 30th United States Society on Dams (USSD) Annual

Temple, D.M.		Conference, 12–16th April 2010, Sacramento, CA.
Harrison, T.A.	1995	*Formwork Striking Times – Criteria, Prediction and Methods of Assessment,* Report 136. CIRIA, London.
Hartford, D.N.D. and Baecher, G.B.	2004	*Risk and Uncertainty in Dam Safety.* CEA Technologies Dam Safety Interest Group. Thomas Telford, London.
Head, K.H.	1982	*Manual of Soil Laboratory Testing*. Vol 2. 2nd Edition. Pentech Press, London
Healy, P.R.	1984	*Construction over Abandoned Mine Workings*. CIRIA Special Publication 32. CIRIA, London.
Hewlett, H.W.M., Boorman, L.A. and Bramley, M.E.	1987	*Design of Reinforced Grass Waterways.* CIRIA Report 116. CIRIA, London. [2003 reprint. Based on Research Project 352. Replaces CIRIA TN 120 and supersedes CIRIA Technical Note 71.]
Highways Agency	1997	*Design Manual for Roads and Bridges Volume 1. Highway structures: approval procedures and general design. Section 3: General design. Part 14: Design of highway structures in areas of mining subsidence.* Stationary Office, London.
Highways Agency	2012	*Design Manual for Roads and Bridges (DRMB).* Available at: http://www.dft.gov.uk/ha/standards/dmrb/index.htm [Accessed February 2013].
Hoek, E., Carranza-Torres, C.T. and Corkum, B	2002	*Hoek-Brown (rock mass) failure criterion – 2002 edition*. In: Proceedings North American Rock Mechanics Society Meeting in Toronto, July 2002.
HM Nuclear Installations Inspectorate	1979	*Safety Assessment Principles for nuclear power reactors.* Office of Nuclear Regulation, HSE. London.
HM Treasury	2003	*The Green Book. Appraisal and Evaluation in Central Government.* TSO, London. In July 2011, pages dealing with the valuation of non-market goods were updated alongside the release of a Green Book discussion paper, *Valuation Techniques for Social Cost Benefit Analysis: Stated Preference Revealed Preference and Subjective Well-Being Approaches*, by D. Fujiwara and R. Campbell. Available at: http://www.hm-treasury.gov.uk/data_greenbook_news.htm [Accessed February 2013]
HM Treasury	2004	*The Orange Book. Management of Risk – Principles and Concepts.* HMSO, Norwich.
Hobbs, N.B.	1986	Mire morphology and the properties and behaviour of some British and foreign peats. *Quarterly Journal of Engineering Geology and Hydrogeology*, 19, 7-80.
HR Wallingford	1996	*Reservoir Dams: Wave Conditions, Wave Overtopping and Slab Protection.* Report SR459. HR Wallingford,

		Wallingford.
HR Wallingford	2010	*Relating Asset Condition to Flood Risk. Development of Supporting Tools and Techniques*. Report TR179. HR Wallingford, Wallingford.
HSE	1992	*The Tolerability of Risk from Nuclear Power Stations.* HMSO, London.
HSE	2001	*Reducing Risk, Protecting People (R2P2), HSE's decision-making process.* HMSO, Norwich.
HSE	2001–2006	*ALARP Suite of Guidance*. Available at: http://www.hse.gov.uk/risk/theory/alarp.htm [Accessed February 2013].
HSE	2007	*CD212 – Proposals for revised policies to address societal risk around onshore non-nuclear major hazard installations*. Available at: http://www.hse.gov.uk/consult/condocs/cd212.htm [Accessed February 2013].
HSE	2009	*Societal Risk: Initial briefing to Societal Risk Technical Advisory Group*. RR703. HSE Books, Sudbury, Suffolk.
HSE	2010	Failure Rate and Event Data for use within Land Use Planning Risk Assessments. Available at: http://news.hse.gov.uk/2010/03/25/failure-rate-and-event-data-for-use-within-land-use-planning-risk-assessments/ [Accessed February 2013].
HSE	2011	PADHI (Planning Advice for Developments near Hazardous Installations). Available at: http://www.hse.gov.uk/landuseplanning/methodology.htm [Accessed February 2013].
Hughes, A.K., Hewlett, H. and Samuels, P.G.	2000	*Risk Management for UK Reservoirs.* CIRIA Report C542. CIRIA, London.
Hughes, A.K. and Gardiner. K.D.	2004	Portfolio risk assessment in the UK: a perspective. In: *Long-term Benefits and Performances of Dams*, ed. H. Hewlett, Proceedings British Dam Society 13th Biennial Conference, Canterbury, 22–26 June 2004. Thomas Telford, London.
Hughes, D.B., Clarke, B.G. and Money, M.S.	1998	The glacial succession in lowland Northern England. *Quarterly Journal of Engineering Geology and Hydrogeology*, 31, 211-234.
Hunt, J.A.	1959	Design of seawall and breakwaters. *Journal of Waterway, Port, Coastal, & Ocean Engineering*, 85, 123-152.
ICE	1996	*Flood and Reservoir Safety*, 3rd edition. Thomas Telford, London.
ICE	1998	*Application Note for the BRE Engineering Guide to Seismic Risk of Dams in the UK*. ICE, London.

ICOLD	1989	*Exposure of Dam Concrete to Special Aggressive Waters*. ICOLD Bulletin 71. International Commission on Large Dams, Paris.
ICOLD	1991	*Alkali-Aggregate Reaction in Concrete Dams*. ICOLD Bulletin 79. International Commission on Large Dams, Paris.
ICOLD	1993	*Rock Foundations for Dams.* ICOLD Bulletin 88. International Commission on Large Dams, Paris.
ICOLD	1994	*Ageing of Dams and Appurtenant Works. Review and Recommendations*. ICOLD Bulletin 93. International Commission on Large Dams, Paris.
ICOLD	1995	*Dam Failures: Statistical Analysis.* ICOLD Bulletin 99. International Commission on Large Dams, Paris.
ICOLD	1996	*Dams and Related Structures in Cold Climate – Design Guidelines and Case Studies.* ICOLD Bulletin 105. International Commission on Large Dams, Paris.
ICOLD	1997	*Concrete Dams – Control and Treatment of Cracks. Review and Case Histories.* ICOLD Bulletin 107. International Commission on Large Dams, Paris.
ICOLD	1997	*Dams less than 30m high – Cost Savings and Safety Improvements*. ICOLD Bulletin 109. 179pp. International Commission on Large Dams, Paris.
ICOLD	2005	*Risk Assessment in Dam Safety Management: A Reconnaissance of Benefits, Methods and Current Applications*. ICOLD Bulletin 130. International Commission on Large Dams, Paris.
ICOLD	2009	*The Physical Properties of Hardened Conventional Concrete in Dams.* ICOLD Bulletin 145. Preprint bulletin. International Commission on Large Dams, Paris.
ICOLD	2011	*Dam Safety Management: Operational phase of the dam life cycle.* ICOLD Bulletin 154. Preprint bulletin. International Commission on Large Dams, Paris
ICOLD	2013	*Bulletin on Internal Erosion*. International Commission on Large Dams, Paris Final Draft issued to member countries 22 Jan 2013 under Circular letter 1859. Final version to be issued mid 2013.
IEC	2006	Analysis techniques for dependability – reliability block diagram and Boolean methods. IEC 61078. International Electrotechnical Commission, Geneva.
Ito, Y.	1971	Stability of mixed type breakwater – a method of probable sliding distance. *Coastal Engineering in Japan*, 14, 53-61.
Johnson, R.A., Leek, D.S., King, E.S. and Dunne, H.B.	1995	*Underground Service Reservoirs: Waterproofing and Repair Manual.* Report 138. CIRIA, London.

Johnston, T.A., Millmore, J.P., Charles, J.A. and Tedd, P.	1999	*An Engineering Guide to the Safety of Embankment Dams in the United Kingdom*, 2nd edition. BRE, Watford.
Kennard, M.F.	1972	Examples of the internal condition of some old earth dams. *Journal of the Institution of Water Engineers*, 26(3), 135-154.
Kennard, M.F.	1994	Four decades of development of British embankments dams. Geoffrey Binnie Lecture. Supplementary issue, Dams & Reservoirs, Conference Discussion, 8th Biennial Conference, Exeter, 14–17 September 1994. British Dam Society, London.
Kennard, M.F., Hoskins, C.G. and Fletcher, M.	1996a	*Small Embankment Reservoirs: A Comprehensive Guide to the Planning, Design, Construction and Maintenance of Small Embankment Reservoirs for Water Supply and Amenity Use*. CIRIA Report 161. CIRIA, London.
Kennard, M.F., Owens, C.L. and Reader, R.A.	1996b	*Engineering Guide to the Safety of Concrete and Masonry Dam Structures in the UK*. Report 148. CIRIA, London.
Kirkgoz, M.S.	1995	Breaking wave impact on vertical and sloping coastal structures. *Ocean Engineering*, 22(1), 35-48.
Kittle, J.A.	1988	Some geological and engineering characteristics of lodgement tills from the Vale of St Albans, Hertfordshire. *Quarterly Journal of Engineering Geology and Hydrogeology*, 21, 183-199.
Ledbetter, S.R., Hurley, S. and Sheehan, A.	1998	*Sealant Joints in the External Envelope of Buildings: A Guide to Design, Specification and Construction*. Report R 178. CIRIA, London.
Le Guen, J.	2010	Exploration of the Tolerable Risk Guidelines for Levee Systems. In: *Proceedings of the Workshop, Exploration of Tolerable Risk Guidelines for the USACE Levee Safety Program, 17–18 March 2010, Washington DC*. US Army Corps of Engineers, Alexandria, VA.
Li, Y., Gong, J.H., Zhu, J., Ye, L., Song, Y.Q. and Yue, Y.J.	2012	Efficient dam break flood simulation methods for developing a preliminary evacuation plan after the Wenchuan Earthquake. *Natural Hazards and Earth System Sciences*, 12, 97-106.
Maijala, T.	2001	*RESCDAM. Development of Rescue Actions Based on Dam-Break Flood Analysis*. Finnish Environment Institute, Helsinki.
MAFF	1988	*Agricultural Land Classification of England and Wales. Revised Guidelines and Criteria for Grading the Quality of Agricultural Land.* Ministry of Agriculture, Fisheries and Food, London. Available at: http://archive.defra.gov.uk/foodfarm/landmanage/land-use/documents/alc-guidelines-1988.pdf [Accessed

		February 2013].
Mason, P.J.	2010	*Loyne Dam – stability review based on a QRA, event tree approach*. In: Proceedings British Dam Society 16th Biennial Conference, Strathclyde, 23–26 June 2010.
McCann, M.W., Franzini, J.B., Kavazanjian, E. and Shah, H.C.	1985	*Preliminary Safety Evaluation of Existing Dams.* Report Prepared for Federal Emergency Management Agency by The John A. Blume Earthquake Engineering Center, Department of Civil Engineering, Stanford University.
McCann, M.W. and Castra, G.	1998	A framework for applying and conducting risk-based analysis for dams. In: *Proceedings of USCOLD 1998 Annual Lecture*, Buffalo, NY, pp. 115-132. United States Committee on Large Dams, Denver, CO.
McLellan, G.	1976	*Chute spillway sub-drainage and anchorage*. In: Proceedings 12th ICOLD Congress, Mexico, Vol. IV, C.16. pp. 1029-1037. International Commission on Large Dams, Paris.
McClelland, D.M. and Bowles, D.S.	2000	Life-loss estimation: What can we learn from case histories? ANCOLD Bulletin 113:75-91.
Moffat, A.I.B.	2002	*The characteristics of UK puddle clays – a review*. In: Proceedings British Dam Society 12th Biennial Conference, Dublin, pp. 581-601.
Morris, M.W.	2009	*Breach Initiation and Growth: Physical Processes.* FLOODsite Report T06-08-11. Available at: www.floodsite.net [Accessed February 2013].
Morris, M.W.	2012	*Breaching of earth embankments and dams,* PhD thesis. The Open University, Milton Keyes.
Morris, M.W., Hassan, M.A.A.M. and Escarameia, M.	2013	*The Performance of Vegetation on Flood Embankments.* FloodProBE Report WP03-01-10-06. EU FloodProBE Project www.floodprobe.eu
Mostafa, T.M.S.	2003	*Experimental modelling of local scour in cohesive soils.* PhD thesis, University of South Carolina.
Mostafa, T M.S., Imran, J., Chaudhry, M.H. and Kahn, I.B.	2008	Erosion resistance of cohesive soils. *Journal of Hydraulic Research,* 46(6), 777-787.
Munger, D.F., Bowles, D.S., Boyer, D.D., Davis, D.W., Margo, D.A., Moser, D.A., Regan, P.J. and Snorteland, N.	2009	Interim tolerable risk guidelines for the US Army Corps of Engineers dams. In: *Managing Our Water Retention Systems, Proceedings of the 29th Annual USSD Conference*, 20–24 April 2009, Nashville, TN, pp. 1125-1142. United States Society of Dams, Denver, CO.
National Audit Office (NAO)	2007	*Environment Agency. Building and Maintaining River and Coastal Defences in England*. Report by the Comptroller and Auditor General, HC 528 2006-2007. TSO, London.
National Research	1996	*Understanding Risk: Informing Decisions in a Democratic*

Council (NRC)		*Society*. National Academy Press, Washington DC.
NERC	1975	*Flood Studies Report.* Five volumes. National Environment Research Council, London.
New South Wales Government Dams Safety Committee	2006	*Risk Management Policy Framework for Dam Safety*. NSW Dams Safety Committee, Parramatta, NSW, Australia.
Northmore, K.J., Bell, F.G. and Culshaw, M.G.	1999	Some geotechnical properties of the Claygate 'Beds' and Bagshot 'Beds' of south Essex. *Quarterly Journal of Engineering Geology and Hydrogeology*, 32, 215-231
ODPM	2004	Commercial and industrial floor space and rateable value statistics 1998–2004.
Ofwat	2000	*Maintaining Serviceability to Customers*. MD161. Ofwat, London.
Owen, M.W. and Steele, A.A.J.	1988	*Wave Prediction in Reservoirs: Comparison of Available Methods*. Report EX1809. HR Wallingford, Wallingford, Oxfordshire.
Parsons, A.W. and Perry, J.	1985	Slope stability problems in ageing highway earthworks. In: *Slope Stability Problems in Ageing Highway Earthworks*, Proceedings ICE Symposium on Failures in earthworks, 6–7 March 1985, London, pp. 63-78. Thomas Telford, London.
Penning-Rowsell, E.C. and Chatterton, J.B.	1977	*The Benefits of Flood Alleviation: A Manual of Assessment Techniques*. Ashgate Publishing, London.
Penning-Rowsell, E.C., Green, C.H., Thompson, P.M., Coker, A.C., Tunstall, S.M., Richards, C. and Parker, D.J.	1992	*The Economics of Coastal Management: A Manual of Benefit Assessment Techniques*. Belhaven Press, London.
Pickard, Q.	2002	*The Architect's Handbook*. Blackwell Science, Malden, MA.
Quillin, K.	2001	*Modelling Degradation Processes Affecting Concrete.* BRE Research Report. HIS BRE Press, Bracknell.
Reader, R.A., Kennard, M.H. and Hay, J.	1997	*Valves, Pipework and Associated Equipment in Dams*. CIRIA Report 170. CIRIA, London.
Reclamation	1976	*Design of Gravity Dams. Design Manual for Concrete Gravity Dams*. US Bureau of Reclamation, Denver, CO.
Reclamation	1987	*Design of Small Dams*, 3rd edition. US Bureau of Reclamation, Denver, CO.
Reclamation	1999	*A procedure for estimating loss of life caused by dam failure.* DSO-99-06. US Bureau of Reclamation, Denver,

		CO.
Reclamation	2009–2011	*Dam Safety Risk Analysis Best Practices Training Manual* [online]. US Bureau of Reclamation, Denver, CO. Available at: www.usbr.gov/ssle/damsafety/Risk/methodology.html [Accessed February 2013].
Reclamation	2009	Chapter 8. Selective probability and expert elicitation. In: *Dam Safety Risk Analysis Best Practices Training Manual* [online]. US Bureau of Reclamation, Denver, CO. Available at: www.usbr.gov/ssle/damsafety/Risk/methodology.html [Accessed February 2013].
Reclamation	2010a	Chapter 24. Internal erosion and piping risks for embankments. In: *Dam Safety Risk Analysis Best Practices Training Manual* [online]. US Bureau of Reclamation, Denver, CO. Available at: www.usbr.gov/ssle/damsafety/Risk/methodology.html [Accessed February 2013].
Reclamation	2010b	Chapter 31. Qualitative and semi-qualitative assessment. In: *Dam Safety Risk Analysis Best Practices Training Manual* [online]. US Bureau of Reclamation, Denver, CO. Available at: www.usbr.gov/ssle/damsafety/Risk/methodology.html [Accessed February 2013].
Reclamation	2010c	Chapter 1. Potential failure mode identification, description and screening. In: *Dam Safety Risk Analysis Best Practices Training Manual* [online]. US Bureau of Reclamation, Denver, CO. Available at: www.usbr.gov/ssle/damsafety/Risk/methodology.html [Accessed February 2013].
Reclamation	2012	*Identifying, describing and classifying potential failure modes*. Presentation at FERC Risk-Informed Decision Making (RIDM) Risk Level 1 Workshop, 2012. Available at: www.ferc.gov/industries/hydropower/safety/initiatives/risk-informed-decision-making/potential-fail.pdf [Accessed February 2013].
Sayers, P.B., Meadowcroft, I.C., Moody, A., Gouldby B., and Crossman, M.	2001	*Integrating reliable flood modelling with probabilistic analysis in practice.* Proceedings of the Flood and Coastal Management Conference, MAFF, June 2001. Published by MAFF (now Defra) 2001.
Sayers, P.B. and Meadowcroft, I.C.	2005	*RASP – A Heirachy of Risk-based Methods and their Application*. In: Proceedings 40th Defra Flood and Coastal Management Conference, 5–7 July, 2005, York. Defra, London.
Simm, J., Gouldby, B., Sayers, P.,	2008	*Representing Fragility of Flood and Coastal Defence: Getting into the Detail*. Floodrisk Conference Workshop,

Flikweert, J., Wersching, S. and Bramley, M.		2008, Oxford.
Skempton, A.W.	1989	Historical development of British embankment dams to 1960. In: *Clay Barriers for Embankment Dams*, Proceedings ICE Conference, October 1989, pp. 15-52. Thomas Telford, London.
Skinner, H.	2000	The use of historical data in assessing the risks posed by embankment dams. *Dams & Reservoirs*, 10(1), 9-12.
SMEC/RAC	1995	*Review of Headworks.* Final Report. Technical consulting report prepared for the Office of Water Reform, Department of Conservation and Natural Resources, Water Victoria, Victoria, Australia
Spencer, E.	1967	A method of analysis of the stability of embankments assuming parallel inter-slice forces. *Geotechnique,* 17, 11-26.
Sutherland, R.J.M., Humm, D. and Chrimes, M.	2001	*Historic Concrete: The Background to Appraisal.* Thomas Telford, London.
Takahashi, S. Tanimoto, K. and Shimosako, K.	1994	*A proposal of impulsive pressure coefficient for the design of composite breakwaters*. In: Proceedings Hydro-Port '94, Yokosuka, Japan.
Tarrant, F.R., Hopkins, L.A. and Bartlett, J.M.	1994	Inundation mapping for dam failure – Lessons from UK experience. In: *Reservoir Safety and Environment*, Proceedings 8th Biennial British Dam Society Conference, 14–17 September 1994, Exeter, pp. 282-291. Thomas Telford, London.
Taylor, R.K.	1988	Coal measures mudrocks: composition, classification and weathering processes. *Quarterly Journal of Engineering Geology and Hydrogeology,* 21, 85-99.
Tedd, P., Skinner, H.D. and Charles, J.A.	2000	Developments in the British national dams database. In: *Dams 2000*, Proceedings of 11th Biennial British Dam Society Conference, 14–17 June 2000, Bath, pp. 181-189. Thomas Telford, London.
Temple, D.M., Robinson, K.M., Ahring, R.M. and Davis, A.G.	1987	*Stability Design of Grass-Lined Open Channels*. Agricultural Handbook Number 667. US Department of Agriculture, Agricultural Research Service, Washington DC.
Temple, D.M. and Hanson, G.J.	1994	Headcut development in vegetated earth spillways. *Applied engineering in agriculture,* 10(5)**,** 677-682.
Thompson, K.M., Rabouw, R.F. and Cooke, R.M.	2001	The risk of groundling fatalities from unintentional aircraft crashes. *Risk Analysis*, 21(6), 1025-1037.
Trenter, N.A.	1999	*Engineering in Glacial Tills.* Report C504. CIRIA, London.

UKWIR	2011	*Deterioration Rates of Long-Life, Low Probability of Failure Assets: Project Report*. UK Water Industry Research, London.
USACE	1989	*Engineering and Design of Retaining and Flood Walls.* Engineer Manual 1110-2-2502. US Army Corps of Engineers, Washington DC.
USACE	2002	*Ice Engineering*. Engineer Manual 1110-2-1612. US Army Corps of Engineers, Washington DC.
USACE	2006	*Coastal Engineering Manual*. Engineer Manual 1110-2-1100. US Army Corps of Engineers, Washington DC.
USACE	2009	*Internal Erosion Toolbox. A Method for Estimating Probabilities of Failure of Embankment Dams due to Internal Erosion, Best Practices Guidance Document.* US Army Corps of Engineers, Washington DC.
USACE	2010	*Beyond the Factor of Safety: Developing Fragility Curves to Characterise System Reliability*. Report SR-10-1. US Army Corps of Engineers, Washington DC.
US Department of Agriculture, Natural Resources Conservation Service (USDA NRCS)	2012	WinDAM B Software. Available at: http://go.usa.gov/8Oq [Accessed Febriary 2013].
van Damme, M., Morris, M.W. and Hassan, M.A.A.M.	2011	*A New Approach to Rapid Assessment of Breach Driven Embankment Failures.* FRMRC Research Report SWP4.4. Flood Risk Management Research Consortium (www.floodrisk.org.uk).
van Gelder, P.	2008	*Reliability Analysis of Flood Sea Defence Structures and Systems.* FLOODsite Project Report T07-08-01. Available at: www.floodsite.net [Accessed February 2013].
Vaughan, P.R., Hight, D.W., Sodha, V.G. and Walbancke, H.J.	1979	Factors controlling the stability of clay fills in Britain. In: *Clay Fills*, Proceedings ICE Conference, 14–15 November 1978, London, pp. 205-218. Institution of Civil Engineers, London.
Vaughan, P.R.	1994	34th Rankine Lecture. Assumptions, prediction and reality. *Geoetchnique*, 44(4), 573-609.
Walters, R.C.S.	1971	Dam Geology. Butterworths, London.
Whitehead, E., Schiele, M. and Bull, W.	1976	*A Guide to the Use of Grass in Hydraulic Engineering Practice.* Technical Note 71. CIRIA, London. [Superseded by CIRIA R116].
Wroth, C.P.	1978	The correlation of index properties with some basic engineering properties of soils. *Canadian Geology Journal*, 15(2). 137-145.

Legislation

- The Reservoirs Act 1975
- The Town and Country Planning (Environmental Impact Assessment) (England and Wales) Regulations 1999 (Statutory Instrument 1999 No. 293)

Official guidance

- Planning Policy Statement 25, *Development and Flood Risk,* Communities and Local Government, 2006.
- National Planning Policy Framework, 2012

Websites

All links given listed were accessed on 18 February 2013.

Environment Agency: 'What's in your backyard'	http://www.environment-agency.gov.uk/homeandleisure/37793.aspx
Environment Agency: Reservoirs	http://www.environment-agency.gov.uk/business/sectors/118421.aspx
Environment Agency: Reservoir flood maps	http://www.environment-agency.gov.uk/business/sectors/125359.aspx
Natural England	http://www.naturalengland.org.uk
Scottish Natural Heritage	http://gateway.snh.gov.uk/
Countryside Council for Wales	http://www.ccw.gov.uk
English Heritage	http://list.english-heritage.org.uk/
Historic Scotland	http://www.historic-scotland.gov.uk/
Historic Wales (portal)	http://jura.rcahms.gov.uk/NMW/start.jsp
Office of National Statistics	http://www.ons.gov.uk/
Met Office	http://www.metoffice.gov.uk
Department for Environment, Food and Rural Affairs (Defra)	http://www.defra.gov.uk
Department for Transport	http://www.dft.gov.uk/webtag
Royal Society for the Protection of Birds	http://www.rspb.org.uk/
UK Land Registry Services (UKLRS)	http://www.landregistryservices.com/
The British Dam Society	www.britishdams.org
Floodrisk Management Research Consortium (FRMRC2)	http://www.floodrisk.org.uk
EurOtop Manual	http://www.overtopping-manual.com
FloodProBE	www.floodprobe.eu
Centre for Energy Advancement though Technological Advance (CEATI)	http://www.ceati.com/
United States Department of Agriculture (USDA): WINDAM	http://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/water/manage/hydrology/?cid=stelprdb1045404
US Bureau of Reclamation	http://www.usbr.gov/
Association of Dam Safety Officials	http://www.damsafety.org/

List of abbreviations

ACOPs	Approved Codes of Practice
AEP	annual event probability
ALARP	as low as reasonably practicable
ALC	Agricultural Land Class
ANCOLD	Australian National Committee on Large Dams
ANSF	Average non-separated flow
AoSP	Area of Special Protection
APF	annual probability of failure
ASLL	average societal life loss
BAP	Biodiversity Action Plan
BEP	backward erosion piping
BPTM	Best Practices Training Manual [Reclamation]
BRE	Building Research Establishment
BS	British Standard
CBA	cost benefit analysis
CCTV	closed circuit television
CEATI	Centre for Energy Advancement through Technological Innovation
CFD	computational fluid dynamics
CIBS	Chartered Institute of Building Services
CIRIA	Construction Industry Research and Information Association
CMSR	concrete/ masonry dams and service reservoirs (that is, gravity wall dams)
COMAH	Control of Major Accident Hazards Regulations
CPF	cost to prevent a fatality
CS	Condition Score
CWI	Catchment wetness index
Defra	Department for the Environment, Food and Rural Affairs
DC	design capacity
DfT	Department for Transport
DSIG	Dam Safety Interest Group

DSRAM	dam safety risk analysis best practice training manual
D/S	downstream (slope)
EAU	Economic Analysis Unit
EFTEC	Economics for the Environment Consultancy
EIA	Environmental Impact Assessment
EKP	economic key point
FCERM	flood and coastal erosion risk management
FEH	Flood Estimation Handbook
FERC	Federal Energy Regulatory Commission
FHRC	Flood Hazard Research Centre
FM	failure mode
FMEA	failure modes and effects analysis
FMI	failure modes identification
FMs	failure modes
FRMRC	Flood Risk Management Research Consortium
FRS	Floods and Reservoir Safety [ICE 1996]
FSR	Flood Studies Report [NERC 1975]
FWMA	Flood and Water Management Act 2010
GIS	geographical information system
HIV	highest individual vulnerability
HSC	Health and Safety Commission
HSE	Health and Safety Executive
ICE	Institution of Civil Engineers
ICOLD	International Commission on Large Dams
IEC	International Engineering Consortium
IR	individual risk
LLOL	likely loss of life
LNR	Local Nature Reserve
LoF	likelihood of failure
LPA	local planning authority
LUP	land use planning
NAO	National Audit Office

NPD	National Property Database
MCM	Multicoloured Manual [FHRC 2010]
NSD	Nuclear Safety Directorate
NSW DSC	New South Wales Dams Safety Committee
ODPM	Office of the Deputy Prime Minister
Ofwat	The Water Services Regulation Authority
OGL	ordinary ground level
PADHI	Planning Advice for Developments near Hazardous Installations [HSE 2011]
PAR	population at risk
PGA	peak ground acceleration
PF	proportion factor
PFMA	potential failure modes analysis
PFR	Prescribed form of record
PMF	probable maximum flood
POF (or Pf)	probability of failure
QRA	qualitative risk assessment
RAC	RAC Engineers and Economists
RAFT	risk assessment field tool
RARS	Risk Assessment for Reservoir Safety
RASP	Risk Assessment for Systems Planning
RIM	reservoir inundation mapping
RMUKR	Risk Management for UK Reservoirs
R2P2	Also known as ‘reducing risks, protecting people’ (see HSE 2001)
SAAR	Standard average annual rainfall (1941–1970)
SAC	Special Area of Conservation
SLF	Stability Index (for gravity dams – sliding)
SMEC	SMEC Holdings Limited (formerly Snowy Mountains Engineering Corporation)
SPA	Special Protection Area
SPT	Seepage and Piping Toolbox
SR	societal risk
SRI	scaled risk integral

SSSI	Site of Special Scientific Interest
SWL	static water level
ToR	Tolerability of Risk
TWL	top (reservoir) water level
UKWIR	UK Water Industry Research
UNESCO	United Nations Educational, Scientific and Cultural Organisation
USACE	US Army Corps of Engineers
USDA	US Department of Agriculture
VPF	value of preventing a fatality
WFD	Water Framework Directive
WTW	water treatment works

Notation

Symbol	Definition	Unit
A	flow cross-sectional area	m^2
B	base width (gravity structure)	m
B	estimated surface width of valley at estimated water depth	m
C	crest width	m
Cd	drag coefficient	(-)
C%	percent clay	
D	depth	m
d	depth	m
E	erosion rate	$m^3/s/m^2$
Ec	compaction effort	$ft\text{-}lb/ft^3$
H	height of dam crest above spillway	m
H	dam height	m
H	depth of water	m
H	height of reservoir water level above flood plain	m
Kd	erodibility or detachment coefficient	(-)
Kd	erosion rate	$cm^3/N\text{-}s$ or $ft/hr/(lb/ft^2)$
K_p	Coefficient of Passive earth pressure	(-)
L	length of dam across the valley (at selected water level)	m
La	attenuation length factor	(-)
n	normal depth	m
n	Manning's roughness coefficient	(-)
P	probability	(-)
Q	discharge	$m^3/s/m$
q	discharge	$m^3/s/m$
Q/W	discharge/flooded width	m^2/s

Q_p	peak discharge	m^3/s
P	wetted perimeter	(-)
R	ratio between breach area and total dam face area	(-)
R	channel radius (spillway chute)	m
Ru	groundwater level/depth of soil	(-)
S_o	slope along the river valley	(-)
T_h	time period at half discharge	s
T_p	time to peak discharge	s
τ	effective shear stress	kPa
τ_c	critical shear stress	kPa
V	velocity	m/s
V	storage volume of reservoir below level H	m^3
V_C	flow velocity	m/s
V_{CL}	critical flow velocity	m/s
W	channel width (spillway chute)	m
WC%	compaction water content	%
x	distance between zone intersections	m
Yd	dry unit weight	mg/m^3
Yw	unit weight of water	mg/m^3

Glossary

Annual exceedance probability	The estimated probability of a flood of given magnitude occurring or being exceeded in any year. Expressed as, for example, 1 in 100 chance or 1%.
Arch dam	Curved and commonly built with concrete, the arch dam is a structure that is designed so that hydrostatic pressure, presses against the arch, compressing and strengthening the structure as it pushes into its foundation or abutments. Arch–gravity dams are the types of dams that use combined engineering methods of arch dams and gravity dams.
Average societal life loss (ASLL)	Average social impact, comprising the sum of the population at risk and likely fatality rate for each of the groups of receptors considered in the consequence assessment.
Buttress dam	Buttress dams are made from concrete or masonry. They upstream side is water tight and supported by triangular buttresses spaced at intervals on the downstream side. These resist the force of the reservoir water trying to push the dam over. A lot less material is needed in a buttress dam than a gravity dam due to the clear spaces between the buttresses.
Consequence(s)	In relation to risk analysis, the outcome or result of a risk being realized. Impacts in the downstream, as well as other, areas resulting from failure of the dam or its appurtenances. Consequence may be expressed quantitatively (for example monetary value), by category (for example High, Medium, Low) or descriptively.
Current Condition	The condition of structure at the present time, according to the cumulative effects of aging and deterioration, and maintenance over its life to date (see also Intrinsic Condition).
Direct losses	Direct damages are those where the loss is due to direct contact with flood water, such as damage to buildings and their contents. These are tangible when they can be easily specified in monetary terms.
Effects	In the context of failure modes analysis (FMEA and FMECA), this term refers to the consequences for the functioning of a system, such as a dam, of a failure at some point within the system. If the system boundary is the dam, these consequences would be distinguished from the remote consequences, which may also be the result of the same failure, such as loss of life and property damage due to a dam-break wave downstream of the dam.
Event tree	An event tree is a graphical construct that shows the logical sequence of the occurrence of events in, or states of, a system following an initiating event. It is not necessary for the events to be placed in the event tree in order of their sequence of accruing.
Exposure	Quantification of the receptors that may be influenced by a hazard (flood), for example, number of people and their demographics, number and type of properties and so on

External threat	Any action originating outside of the body of the dam, and thus include floods, earthquake, mining, subsidence and so on.
Extreme event	An extreme event is simply an event that has a very low probability of occurrence (that is statistically does not happen very often, although this does not mean that two rare events cannot happen in close succession).
Factor of safety	See ‘Safety coefficient’.
Failure (of a dam)	In the general case, the inability of a dam system, or part thereof, to function as intended. Thus, in terms of performance to fulfil its intended function, the inability of a dam to perform functions such as water supply, prevention of excessive seepage or containment of hazardous substances. In the context of dam safety, failure is generally confined to issues of structural integrity, and in some contexts to the special case of uncontrolled release of the contents of a reservoir through collapse of the dam or some part of it.
Failure mode analysis	A way that failure can occur, described by the means by which element or component failures must occur to cause loss of the sub-system or system function.
Failure mode and effects analysis (FMEA)	An inductive method of analysis where particular faults or initiating conditions are postulated and the analysis reveals the full range of effects of the fault or the initiating condition on the system. FMEA can be extended to perform what is called failure modes, effects and criticality analysis (FMECA). In a FMECA, each failure mode identified is ranked according to the combined influence of its likelihood of occurrence and the severity of its consequences.
Failure mode or mechanism	A way that failure can occur, described by the means by which element or component failures must occur to cause loss of the sub-system or system function.
Fault tree	A fault tree is a common method to analyse failure probabilities of complex systems. The fault tree is a tool for linking various failure mechanisms leading to an expression of the probability of system failure.
Floods Directive	A European Community Directive (2007/60/EC) of the European Parliament and Council, designed to establish a framework for the assessment and management of flood risks aiming at the reduction of the adverse consequences associated with floods on human health, the environment, cultural heritage, economic activity and infrastructure. The three main requirements of the Directive are the development of Preliminary Flood Risk Assessments (by December 2011), flood hazard and risk maps (by December 2013), and flood risk management plans (by December 2015).
Flood Zone	A geographic area within which the flood risk is in a particular range, as defined within Planning Policy Statement 25 (PPS25): Development and Flood Risk.
F-N curve	Curves that relate F (the probability per year of causing N or more fatalities) to N. This is the complementary cumulative distribution function. Such curves may be used to express societal risk criteria

	and to describe the safety levels of particular facilities.
Fragility	A function that defines the probability of failure as a function of an applied load level. A particular form of the more general system response (qv).
Gravity dam	A gravity dam is made from concrete or masonry, or sometimes both. It is called a gravity dam because gravity holds it down to the ground stopping the water in the reservoir pushing it over.
Hazard	A physical event, phenomenon or human activity with the potential to result in harm. A hazard does not necessarily lead to harm. A source of potential harm or a situation with a potential to cause loss.
Highest individual vulnerability	Individual with highest vulnerability downstream of any individual dam. Used in assessing whether risks are tolerable.
Hypothetical person	Defined in *Reducing Risk, Protecting People,* as part of assessing risk to the public. An hypothetical person is the term used to describe an individual who is in some fixed relation to the hazard, for example the person most exposed to it, or a person living at some fixed point or with some assumed pattern of life. For example, occupational exposure to chemicals, entailing adverse consequences after repeated exposure for long periods, is often controlled by considering the exposure of an hypothetical person who is in good health and works exactly 40 hours a week
Individual vulnerability	Risk to a hypothetical person downstream of the dam. Product of the exposure (% of the time they are present), the likelihood of death if the dam failed (fatality rate)
Individual risk	Risk to a hypothetical person downstream of the dam. Product of individual vulnerability and annual probability of failure of the dam.
Indirect losses	Indirect damages are losses that occur due to the interruption of some activity by the flood, for example the loss of production due to business interruption in and outside the affected area or traffic disruption. These also include the extra costs of emergency and other actions taken to prevent flood damage and other losses. These are tangible when they can be specified in monetary terms.
Internal threat	Pre-existing internal flaws or process which lead to deterioration of a dam sufficient to be the root cause of failure. This may lead directly to failure under constant load, or may weaken the dam to such an extent that it fails rapidly when subject to a change in external load.
Intrinsic condition	The condition afforded a structure by its very nature according to its type and quality of materials, method of construction and geometry. (See also Current Condition)
Joint probability	The probability that two or more variables will assume certain values simultaneously or within particular time intervals.
Likelihood	A general concept relating to the chance of an event occurring. Likelihood is generally expressed as a probability or a frequency.
Likely loss of life	Likely loss of life, this term has been superseded in this guide by

(LLOL)	average societal life loss (ASLL).
Limit state	The boundary between safety and failure.
Overtopping	Water flowing over the top of the dam, other than over spillweirs or crests.
Planning Policy Statement (PPS)	A statement of spatial planning policy issued by central government (generally to replace older Planning Policy Guidance notes).
Probable maximum flood (PMF)	The flood hydrograph resulting from Probable Maximum Precipitation (PMP) and, where applicable, snowmelt, coupled with the worst flood producing catchment conditions that can be realistically expected in the prevailing meteorological conditions.
Probability	A measure of the degree of confidence in a prediction, as dictated by the evidence, concerning the nature of an uncertain quantity or the occurrence of an uncertain future event. It is an estimate of the likelihood of the magnitude of the uncertain quantity, or the likelihood of the occurrence of the uncertain future event. This measure has a value between zero (impossibility) and 1.0 (certainty). There are two main interpretations: • Statistical – frequency or fraction. The outcome of a repetitive experiment of some kind like flipping coins. It includes also the idea of population variability. Such a number is called an 'objective' probability because it exists in the real world and is in principle measurable by doing the experiment. • Subjective probability – Quantified measure of belief, judgement, or confidence in the likelihood of an outcome, obtained by considering all available information honestly, fairly, and with a minimum of bias. Subjective probability is affected by the state of understanding of a process, judgement regarding an evaluation, or the quality and quantity of information. It may change over time as the state of knowledge changes.
Qualitative risk analysis	An analysis, which uses word form, descriptive or numeric rating scales to describe the magnitude of potential consequences and the likelihood that those consequences will occur.
Quantitative risk analysis	An analysis based on numerical values of the potential consequences and likelihood, the intention being that such values are a valid representation of the actual magnitude of the consequences and the probability of the various scenarios which are examined.
Regulated Reservoir	A reservoir that must be registered under the Reservoirs Act 1975.
Reliability analysis	Likelihood of successful performance of a given project element. It may be measured on an annualised basis or for some specified time period of interest or, for example, in the case of spillway gates, on a per demand basis. Mathematically, Reliability = 1 – Probability of

	failure.
Residual risk	The remaining level of risk at any time before, during and after a program of risk mitigation measures has been taken.
Resilience	The ability of a system/community/society/defence to react to and recover from the damaging effect of realised hazards.
Return period	The expected (mean) time (usually in years) between the exceedance of a particular extreme threshold. Return period is traditionally used to express the frequency of occurrence of an event, although it is often misunderstood as being a probability of occurrence.
Risk	The combination of the chance of a particular event (for example a flood), with the impact that the event would cause if it occurred. Risk therefore has two components – the chance (or probability) of an event occurring, and the impact (or consequence) associated with that event.
Risk analysis	The use of available information to estimate the risk to individuals or populations, property or the environment, from hazards. Risk analyses generally contain the following steps: scope definition, hazard identification, and risk estimation. Consistent with the common dictionary definition of analysis, viz. 'A detailed examination of anything complex made in order to understand its nature or to determine its essential features', risk analysis involves the disaggregation or decomposition of the dam system and sources of risk into their fundamental parts.
Risk assessment	The process of making a decision recommendation on whether existing risks are tolerable and present risk control measures are adequate, and if not, whether alternative risk control measures are justified or will be implemented. Risk assessment incorporates the risk analysis and risk evaluation phases. Consistent with the common dictionary definition of assessment, viz. 'To analyse critically and judge definitively the nature, significance, status or merit of…[risk]', risk assessment is a decision-making process, often sub-optimal between competing interests, that results in a statement that the risks are, or are not, being adequately controlled. Risk assessment involves the analysis, evaluation and decision about the management of risk and all parties must recognize that the adverse consequences might materialise and owners will be required to deal effectively with consequences of the failure event.
Risk control	The implementation and enforcement of actions to control risk, and the periodic re-evaluation of the effectiveness of these actions.
Risk estimation	The process of quantifying probabilities and consequences for all significant failure modes.
Risk evaluation	The process of examining and judging the significance of risk. The risk evaluation stage is the point at which values (societal, regulatory, legal and owners) and value judgements enter the decision process, explicitly or implicitly, by including consideration of the importance of the estimated risks and the associated social, environmental, economic, and other consequences, in order to

	identify and evaluate a range of alternatives for managing the risks.
Risk identification	The process of determining what can go wrong, why and how.
Risk management	The systematic application of management policies, procedures and practices to the tasks of identifying, analysing, assessing, mitigating and monitoring risk.
Risk mitigation	A selective application of appropriate techniques and management principles to reduce either likelihood of an occurrence or its adverse consequences, or both.
Risk-based decision-making	Decision–making, which has as a main input the results of risk assessment. It involves a balancing of social and other benefits and the residual risks.
Safety coefficient	In structural and other engineering systems, the ratio of system resistance to the peak design loads, often calculated in accordance with established rules.
Scenario	A situation given a predicted sequence of events.
Sensitivity analysis	An analysis to determine the rate at which an output parameter varies, given unit change in one or more input parameters. Sensitivity can be visualised as the slope of the output parameter graph or surface at the relevant input parameter value or values.
Service reservoir	A service reservoir is a water storage container for clean water after it has been treated, and before it is piped to end users. These containers are covered, and designed to keep water safe from contamination. Their main purpose is to provide a reserve within the water supply system so that water supplies can be maintained across periods of varying demand.
System	A system is a defined entity that consists of identifiable, interacting discrete elements. It is an orderly arrangement of these elements (for example, area within spatial boundaries, structures, mechanical and electrical equipment items, and operators) designed to show the interactions between the various elements in the performance of the system function. For simplicity, the general term system will be used without distinction between various levels of systems.
System response	How a dam responds, expressed as a conditional probability of failure, to a given scenario of applied loads and concurrent conditions. See also fragility curve.
Tolerable risk	(tolerability) Refers to willingness to live with a risk to secure certain benefits and in the confidence that it is being properly controlled. To tolerate a risk means that we do not regard it as negligible, or something we might ignore, but rather as something we need to keep under review, and reduce still further if and as we can. Tolerability does not mean acceptability.
Ultimate limit state	Limiting condition beyond which a structure or element is assumed to become structurally unfit for its purpose.
Uncertainty	A general concept that reflects our lack of sureness about someone

	or something, ranging from just short of complete sureness to an almost complete lack of conviction about an outcome. In the context of dam safety, uncertainty can be attributed to (i) inherent variability in natural properties and events, and (ii) incomplete knowledge of parameters and the relationships between input and output values.
Vulnerability	Characteristic of a system that describes its potential to be harmed. This can be considered as a combination of susceptibility and value.

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