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Guide to drawdown capacity for reservoir safety and emergency planning

SC130001 Volume 2 – supplementary and background information

Flood and Coastal Erosion Risk Management Research and Development Programme

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1 Introduction

A key factor in avoiding and minimising the impact of a catastrophic dam failure is the ability to draw a reservoir down in the event of an emergency. Reservoir drawdown is also important to allow inspection and maintenance of the structures retaining a reservoir.

It is a legal requirement under the Reservoirs Act 1975 (Schedule 5 to Statutory Instrument 2013 No. 1677) that inspecting engineers should review the 'efficiency of the scour pipe or discharge culvert or other means of lowering the water in … the reservoir' during statutory inspections under Section 10 of the Act, with the inspections being carried out at least every 10 years. Similar requirements are included in Welsh legislation and it is good practice in Scotland. In the past there has been no universally accepted approach which could be applied by reservoir owners and inspecting engineers to assess what constitutes an adequate rate of drawdown.

Volume 1 of this guide therefore provides guidance on a consistent methodology for assessing the adequacy of existing drawdown capacity at reservoirs in the UK. The guidance is not statutory; however, it is recommended that where an engineer feels it is right to depart from the general principles, the reasons for the departure should be presented in the assessment.

This volume of the document, Volume 2, provides background and supplementary information relating to the derivation of the guidance in Volume 1 as follows:

- Section 2 presents a review of the approaches currently taken to evaluating drawdown capacity both in the UK and internationally.
- Section 3 describes the findings from industry consultation carried out as part of this project.
- Sections 4 to 7 describe the scoping studies carried out to develop the overall frameworks and identify the factors which should be taken into account in the guide. **In some cases the approach adopted in the final version of the guide was refined following the studies reported here and where this is the case it is noted in the relevant section.**
- Having determined that the time it would take for the dam to fail is a key parameter in deciding what is an appropriate drawdown rate, Section 8 reviews methods for predicting the time to failure, particularly in relation to internal erosion. The outcome from this work led to the theoretical drawdown rates required to avert internal erosion, which are provided in Volume 1.

Note that in various places throughout this volume, drawdown is expressed in relation to dam height but in Volume 1 of the guide this has been more accurately refined to maximum reservoir depth.

2 Review of current drawdown standards

2.1 Data sources

A survey of available information has been undertaken in order to understand the current practice with regard to emergency drawdown capacity of reservoirs and assess the need for new industry guidance. Four sources of data were explored as follows:

- A review of published literature covering relevant published papers and documented national standards from the UK and internationally.
- Reservoir inspection reports under Section 10 of the Reservoirs Act, held on the Environment Agency's database in Exeter, were screened to extract information relating to drawdown capacity. This follows on from work carried out by Alan Warren (Warren 2012) with a total of 197 Section 10 reports being screened.
- A number of major UK reservoir owners were contacted directly to enquire about their current standards for drawdown capacity.
- Drawdown rates achieved during published incidents.

Sections 2.2 to 2.4 describe how data has been collected and discuss the findings from each data source. Sections 2.5 and 2.6 summarise the findings from all of the data collection methods, in relation to the UK and internationally.

2.2 Literature review

2.2.1 Methodology

A literature review was previously carried out in 2005 and published in Table 4.9 of the *Engineering guide to emergency planning for UK reservoirs* (Defra 2006). This identified three international standards and one UK standard relating to the required capacity for the rate of drawdown.

The current literature review has expanded this previous work. A search for relevant published papers and documented national standards was carried out using the following sources:

- Institution of Civil Engineers (ICE) virtual library
- Proceedings from the British Dam Society (BDS) biennial conferences
- Papers published in the *Dams and Reservoirs* journal
- The bibliography of British Dams, which is a list of published works with information on dams in Great Britain. The bibliography is available to BDS members on the BDS website.
- Google search
- Bureau of Indian Standards
- Construction information service

- International Commission on Large Dams (ICOLD) bulletins
- Australian National Commission on Large Dams (ANCOLD) website
- United States, Department of the Interior, Bureau of Reclamation (USBR) website
- References compiled during the preparation of the *Guide to risk assessment for UK reservoir safety management* (RARS; Environment Agency 2013)

In each case, the sources were searched using a series of key words such as 'emergency drawdown', 'drawdown', 'outlet capacity', 'bottom outlet', 'scour outlet'.

2.2.2 Findings

A summary of the content from all sources with relevant information is presented in Appendix A and the findings are summarised below. Table 2.1 summarises the standards identified from the literature review.

While the *Engineering guide to emergency planning* (Defra 2006) gives guidance on what should be considered in determining a drawdown capacity it does not specify an actual drawdown rate and hence has not been included in the table.

International standards

Three international drawdown standards had previously been identified in the *Engineering guide to emergency planning* (Defra 2006). The current literature review found three further international drawdown standards, from India (Bureau of Indian Standards, 2004), the US Army Corps of Engineers (USACE, 2016) and from Norway (FAO 2009). A comparison of the international standards is given in Section 2.6.

Papers from Australia confirm that there are no current Australian (ANCOLD) guidelines for the appropriate sizing of emergency low-level outlet works and USBR (1990) remains the primary reference for Australian dam owners undertaking these assessments (stated in Johnson et al. 2010).

No references to a specific drawdown rate were found in any of the ICOLD bulletins.

UK standards

Eight new drawdown standards were found relating to UK reservoirs in addition to the criterion proposed by Prentice (2005) previously identified in the *Engineering guide to emergency planning* (Defra 2006). The standards are expressed in different ways which makes direct comparison difficult. Therefore, an attempt has been made to non-dimensionalise them and express the required drawdown rate in terms of percentage of reservoir depth per day and all nine standards are compared in this way in Table 2.2. The assumptions made in this process are detailed in the notes to the table. Some of the standards define an initial drawdown rate in the first 24 hours and these are generally expressed in terms of depth per day. Other standards are defined as an overall 'global' drawdown (e.g. a proportion of the reservoir height or volume to be emptied over several days). A few of the standards give values for both of these criteria as shown in Table 2.2.

Table 2.2 highlights that there is no common approach to designing reservoir drawdown capacity in the UK and reservoir owners have adopted a wide range of different standards.

Table 2.1 Summary of UK and international standards found in literature review

No.	Drawdown criteria (see note 1)	Inflows	References	Organisations which use this standard (grey highlight = International)
	Proportion of dam height			
1	Drawdown to **75%** height in **3 days**	$0.5m^3/s$	Welbank et al. (2008)	Wessex Water
2	Drawdown to **75%** height in **10–20 days**. Longer for lower risk reservoirs (USBR criterion)	Highest mean monthly inflows for the duration of the evacuation period	USBR (1990)	USBR, Melbourne Water (in combination with 'ALARP' - as low as reasonably practicable - risk assessments)
3	Drawdown to **90%** of depth in **7–10 days** (only applies to reservoirs >$6.2Mm^3$)	Nil	Babbit and Mraz (1999)	State of California (for larger reservoirs)
4	Drawdown to: **75%** height in **20–50 days,** 50% height in 40–70 days, 25% height in 80–100 days		Bureau of Indian Standards (2004)	India
5	Drawdown to higher of 6.1m depth, or to 10% height in 4 months	Average flow of the highest consecutive 4-month period	USACE (2016)	USACE
	Height per day			
6	1m/day		Philpott et al. (2008)	Thames Water
		Q_{10}	Crook et al. (2010)	Other
7	300mm/day+5H+8640Q_{10}/a (Hinks' formula)	Q_{10}	Hinks (2009)	Private individual
8	Site-specific drawdown rate, minimum 0.5m/day	None	Northern Ireland Water (2014)	Northern Ireland Water
	Proportion of dam volume			
9	Drawdown to **50%** capacity in **7 days** (only applies to reservoirs <$6.2Mm^3$)	Nil	Babbit and Mraz (1999)	State of California (for smaller reservoirs)
10	Drawdown to **50%** capacity in **8 days**		Combelles at al. (1985)	France
11	Drawdown to **50%** volume in **3–9 days** depending on consequence class	Winter daily mean inflow	Brown (2009)	Canal & River Trust (CRT) and Scottish canals
12	Drawdown to **50%** capacity in **10 days** (20 days for non-impounding/small relative catchment)	Nil	Tam and Humphrey (2012)	Anglian Water
13	Drawdown to **25%** capacity in **28 days**	Winter 28-day peak	Prentice (2005)	Northumbrian Water
	Other or combined			
14	Hinks' formula/CRT hybrid (Hinks' formula combined with CRT , with relaxations and exemptions related to dam and downstream parameters)	Q_{10}	Mann et al. (2014)	Scottish Water
15	Hinks' formula/hybrid (The more conservative of (i) Hinks' formula, (ii) Drawdown to 75% height in 14 days for dam category A/B and 30 days for dam category C/D) (see Note 2)	Hinks = Q_{10}	Chesterton et al. (2014)	Severn Trent Water

Notes:
1. For each type of criterion, standards are presented in approximate order of required capacity (most onerous first).
2. Dam category is defined in ICE (2015).

Table 2.2 UK drawdown rates identified in literature review

Organisation	Drawdown expressed as % reservoir height per day (Note 3)		Comments
	Initial drawdown rate	Global drawdown rate	
Thames Water	13% (Note 7)		
Scottish Water	3.2% based on Hinks (Note 4)	Cat A: 5.4–9.0% C and D: 1.4%	
Wessex Water		8.3%	
Canal & River Trust		A1: 4.1% A2: 3.0% B: 2.3%	
Anglian Water		2.1% (1.0% for non-impounding/small relative catchment)	
Northumbrian Water		1.3%	Previously identified in Defra (2006)
Northern Ireland Water	2.6–2.7%		See Appendix A1 Site specific 0.5–0.7m/day
Severn Trent Water	2.7% based on Hinks (Note 5)	(i): 2.7% (Hinks) (ii): 0.8–1.8%	
UK individual (Jonathan Hinks)	3.4% (Note 6)	-	

Notes:

1. The original criteria may be expressed differently in the original references but this table attempts to express the criteria in a common way to aid comparison.
2. Initial drawdown rates will reduce as the reservoir lowers.
3. Where the criterion applies to a specific dam or a specific water company's stock of dams then the daily drawdown criterion has been expressed as a percentage of the dam height or median dam height. The median dam height for each company's stock of dams has been determined from the BRE Dam Register (1994). Where the criterion is defined in terms of reservoir volume, it is assumed that 50% reservoir volume equates to 79% reservoir height and 75% volume equates to 91% reservoir height (assuming a simplified cone-shaped reservoir basin).
4. Based on Scottish Water average dam height of 11.1m.
5. Based on Severn Trent Water average dam height of 13.4m.
6. Based on UK average dam height of 10.3m.
7. Based on Thames Water average dam height of 7.6m.

When these values are normalised as a percentage of reservoir height per day the rate of drawdown varies between 2.6 and 13% height per day as an initial drawdown rate and between 0.8 and 9% height per day in terms of a global drawdown rate. This range represents a significant variation in outlet sizes.

Further UK standards are discussed in Sections 2.3 and 2.4.

Lessons learned from incidents and exercises

Failures and incidents which have occurred at British and overseas dams between 1800 and 2012 are summarised in CIRIA Report SP167 (CIRIA 2014). The report describes 11 incidents in particular, including 3 overseas, where the ability to draw the reservoir down averted disaster. The drawdown rate in these cases varied between 0.8 and 1.7m per day which equates to 1.4 to 11.3% reservoir height per day (see Appendix A.2 and summary in Table 2.3). The depth of drawdown was only stated in three cases and ranged from 3m to 9.3m; hence it is not possible to be certain whether these rates are initial drawdown rates or global rates but they are likely to be the latter. This highlights the importance of adequate drawdown facilities and provides a useful benchmark for required drawdown capacity.

Further evaluation of actual failure incidents is given in Section 8.6.

References to drawdown are made for five further incidents in CIRIA (2014) in terms of problems which arose associated with reservoir drawdown. The problems included dam settlement, cracking of puddle clay cores and slope instability.

Two papers were found which described emergency drawdown exercises carried out by the Canal & River Trust (Brown et al. 2010, Windsor 2012). They demonstrated that it was feasible to mobilise $1m^3/s$ mobile pumping capacity within 24 hours using a framework contractor whose remit included capital delivery projects and emergency works.

Table 2.3 Summary of reported incidents where drawdown averted failure (CIRIA 2014)

Reservoir	Owner	Dam height	Details of emergency drawdown	Drawdown expressed as % reservoir height/day	Capacity of temporary installation
Anglezarke (Heapey)	United Utilities	14m	Emergency drawdown carried out in 1997 which averted disaster. 3m drawdown but rate not stated.		Emergency pumps were delivered to site and used for the first 48 hours.
Balderhead	Northumbrian Water	48m	Emergency drawdown in 1967 by 9.2m following development of two sinkholes on crest. This averted failure.		
Boltby	Yorkshire Water	19m	Emergency drawdown in 2005 averted disaster, following damage caused by a 1 in 10,000 year flood event. Drawdown rate not stated.		Temporary pumps used in combination with scour facility and draw-off.
Fontenelle	USBR	40m	In 1965, failure was averted by rapid reservoir drawdown at a rate of 1.2m/day. Leakage rates reached 600l/s.	3.0	
Greenbooth	United Utilities	35m	Emergency drawdown in 1983 by 9.3m at average rate of 1.2m/day following development of a depression on the crest. This averted failure.	3.4	
Lambieletham	Fife Council	15m	Emergency drawdown in 1984 by 1.7m/day averted disaster.	11.3	Pumps were brought onto the site by helicopter.
Lluest Wen	Welsh Water	24m	An emergency drawdown was carried out in 1970 at a rate somewhere between 0.8m and 1.6m/day.	3.3–6.6	A large number of pumps, some of which were positioned by helicopter, were used.
Martin Gonzalo		54m	In 1987 emergency drawdown was instigated at a rate of 1.5m/day which averted major failure. Leakage rates reached 1,000l/s.	2.8	
Peruca		63m	Following five hostile explosions in the inspection gallery in 1993, the water level was lowered at a rate of about 0.9m/day and failure was averted. A very 'near miss' as leakage rates reached 600l/s.	1.4	
Ulley	Yorkshire Water	16m	The ability to lower the reservoir quickly in 2007 helped prevent a disaster. The drawdown rate was quoted in the records as 18,000m^3/day equating to 150mm/day (Hinks et al. 2008).	0.9	Reliance on temporary pumps. Emergency pumps from the fire service and additional pumps provided later.
Upper Rivington	United Utilities	14m	Achieved a drawdown rate of 1m/day and averted failure in 2002.	7.1	

2.3 Environment Agency database of Section 10 inspection reports

2.3.1 Methodology

Drawdown standards which have been applied during periodic safety reviews at individual reservoirs have been evaluated by screening a sample of inspection reports carried out under Section 10 of the Reservoirs Act held on the Environment Agency's database in Exeter.

Previous work carried out by Warren (2012) summarised recommendations made in the interests of safety from Section 10 reports between 2004 and March 2012. The paper identified 15 reports which included recommendations in the interests of safety for bottom outlet capacity improvements and/or bottom outlet relining. These 15 reports were obtained and information relating to drawdown standards was also extracted from these.

All Section 10 reports from March 2012 to 2014 were screened and examined further if they contained any recommendations in the interests of safety (but not necessarily relating to drawdown). This was considered to be a reasonable means of sampling the database to extract approaches taken to evaluation of the adequacy of drawdown capacity. Any reports that contained recommendations relating to drawdown rate were recorded in the table in Appendix B. In addition the majority of the other reports were interrogated to identify any relevant guidance or references to drawdown capacity.

This method of screening and data recording means that all Section 10 reports containing Matters in the Interests of Safety (MIOS) relating to drawdown capacity between 2004 and 2014 have been captured and information on drawdown capacity obtained.

2.3.2 Summary statistics

Detailed findings from the review are tabulated in Appendix B1. In total, 197 inspection reports were screened. The vast majority of these were from England and Wales but two were from Scotland. Out of the 197 reports, 62 reports (31% of them) were logged as either containing MIOS under Section 10 of the Reservoirs Act 1975, relating to drawdown rate or some other comment on what was considered to be an adequate rate. The other 69% of the reports contained no mention of specific drawdown capacity or criteria.

Of the 62 reports where drawdown capability was assessed, only 26 stated a specific criterion. The criteria in these 26 reports are presented in full in Appendix B2 and summarised in Table 2.4. The drawdown criteria have been normalised as a percentage of dam height per day and compared in Figures 2.1 and 2.2.

The other 135 reports sampled did not include a detailed assessment of drawdown capacity. In some cases there was no mention of drawdown capacity, in other cases the reports simply stated that the existing facilities or lack of facilities were acceptable with no justification in terms of drawdown capability.

The fact that only 31% of the reports sampled included an assessment of drawdown adequacy, and that only 13% of the reports actually referred to a drawdown standard, highlighted the need for this industry guidance to be developed.

Table 2.4 Standards for UK drawdown rate extracted from inspection reports

Standard	Drawdown criteria		% reservoir height/ day	No. of Section 10 reports which used this standard	Percent-age of total reports	Comments
	Initial rate	Global rate				
Prentice 2005 (Defra 2006)		Draw the reservoir down to 70% depth in about 6 days	1.9	1	3.8	Inflow not stated
Hinks' formula			-	3	7.7	Q_{10} inflow
Anglia Water rule		50% volume within 10 days	2.1	2	7.7	No inflow or not stated
CRT rule		50% volume within 5 days	4.1	2	11.5	Winter daily mean inflow
		50% volume – duration not stated	-	1		Inflow not stated
Not referenced	3% height/ day		3	2	7.7	Scottish Water reports Q_{10} inflow
	500mm/day		3.3–7.1	3	61.6	One believed to be Hinks'. One allows 100l/s inflow, others not stated
	300mm/day		3.8–6.0	4		Q_{10} inflow in one case, others nil or not stated
	320mm/day		9.1	1		Not stated
	1m/day		4.3–18.9	5		No inflow (sometimes as non-impounding)
		50% volume in 20 days, (emptying in 30 days)	1.03 (3.3)	1		No inflow
		25% of reservoir volume in 28 days	0.33	1		Inflow not stated
			Total	**26**	**100%**	

Figures 2.1 and 2.2 show the correlation between adopted drawdown rate (expressed as a percentage of dam height) and dam height. The correlation shows that the proportion drawdown is generally less for higher dams because similar drawdown criteria are currently being used regardless of dam height. Figure 2.3 shows how the adopted drawdown rates (again expressed as

a percentage of dam height) vary by dam category. The trend indicates a higher drawdown rate for lower category dams which is the reverse of what would be expected but is because, within the sample, the lower category dams had a lower average height and thus the relationship with dam height explained above dominates.

Figure 2.1 Adopted drawdown rates in Section 10 reports by dam height

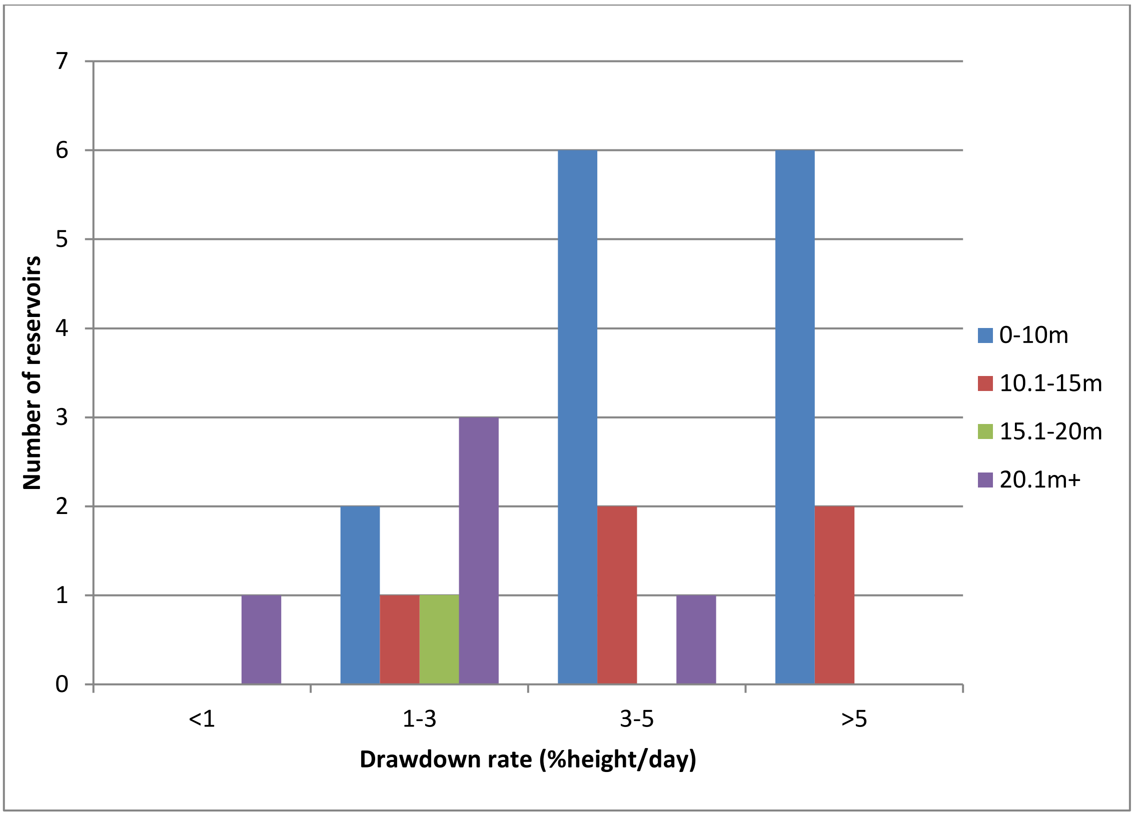


Figure 2.2 Adopted drawdown rates in Section 10 reports by dam height

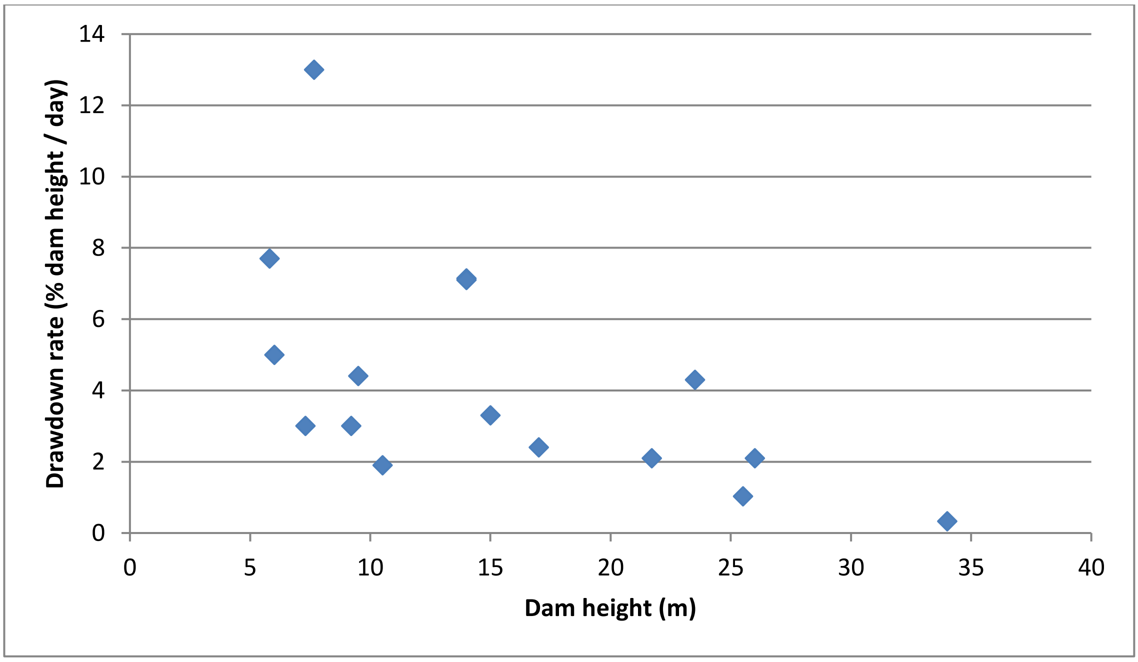
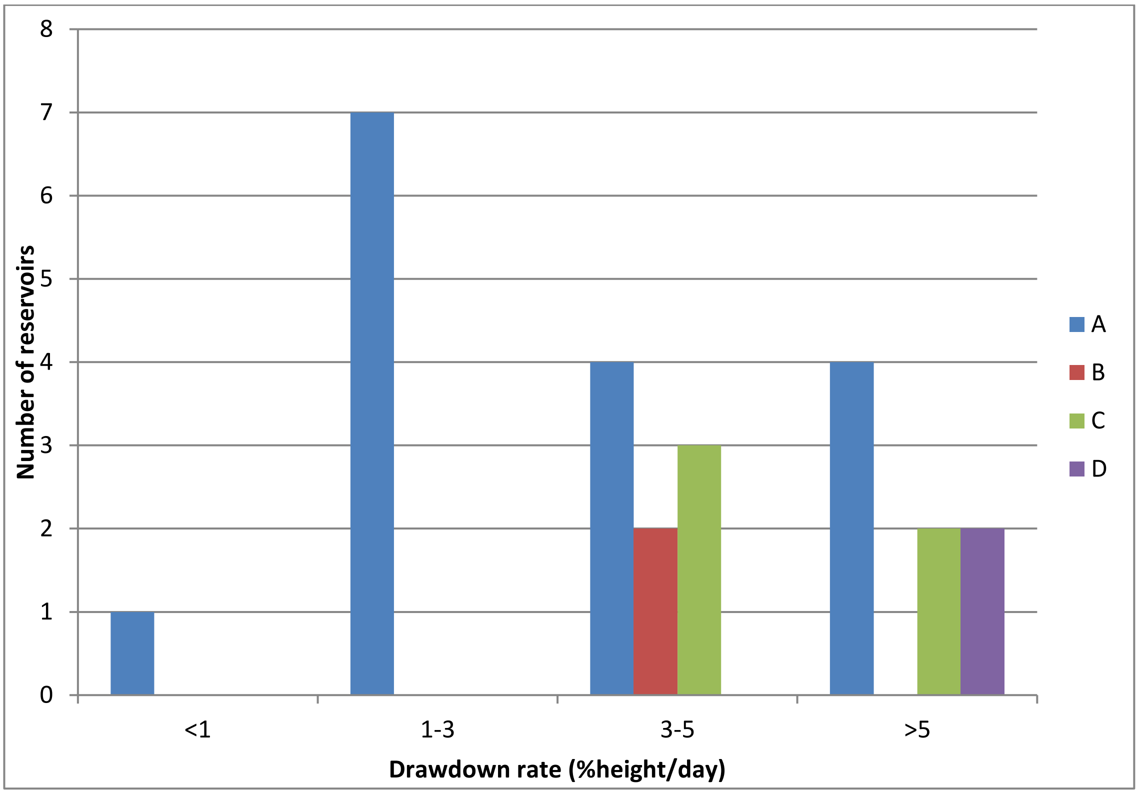


Figure 2.3 Adopted drawdown rates in Section 10 reports by dam category



Out of the 26 reports where drawdown capability was assessed, 77% made no allowance for dealing with reservoir inflows during drawdown, or at least did not state an allowance. In the six reports where inflows were explicitly considered they were typically 'wet day' flows rather than flood flows (see Table 2.5). The general lack of consideration of inflows was taken into account in developing Volume 1 of the current guidance.

Table 2.5 Inflow assumptions in setting drawdown rate from Section 10 reports

Inflow criterion	Number	% total	Comments
No inflow	5	19.3	
No inflow – non-impounding	1	3.8	
Winter mean daily	2	7.7	1 further believed to have used this (included in 'not stated')
Flow exceeded 10% of days a year (Q_{10})	3	11.6	
100l/s	1	3.8	
Not stated	13	50	
Not stated (non-impounding)	1	3.8	
Total	**26**	**100**	

The recording of which reservoirs had a low-level outlet started part-way through the screening of the Section 10 reports and hence this was only identified for 118 of the 197 reservoirs. The results of this check are summarised in Table 2.6. For nearly one-third of the reports checked it was unclear whether there was a low-level outlet or not, either because the report did not contain a dedicated section on the means of drawdown or it was unclear. However, the sample where it was clearly mentioned suggests that approximately two-thirds of reservoirs do have a low-level outlet.

For the literature review a low-level outlet was assumed to be any outlet that could discharge stored water from the base (or significantly below the top water level) of the reservoir.

Table 2.6 Summary of low-level outlet provision from Section 10 reports

Criterion	Number of reservoirs	% total	% excluding unknown values	Comment
No low-level outlet	23	19	34	
Low-level outlet	63	53	66	
Unknown	32	27		Either not clear or the report does not contain a dedicated section on the means of lowering the reservoir
Total	**118**	**100**	**100**	

2.4 Responses from major reservoir owners

In order to further research the reservoir drawdown standards currently adopted in the UK, letters were sent to 14 major dam owners enquiring about their approach. An example of the letter is included in Appendix C1. Responses were received from seven organisations and these are detailed in Appendix C2 and summarised in Table 2.7.

Drawdown standards for the following other water companies have been identified from the literature review (see Tables 2.1 and 2.2):

- Scottish Water
- Northumbrian Water
- Anglian Water
- Wessex Water
- Northern Ireland Water

Table 2.7 Brief summary of responses on drawdown standards from major UK dam owners

Organisation	Drawdown rate	Criterion expressed in approx. % reservoir height/day (note 1)	Inflows	Use of temporary pumps
Thames Water (note 2)	1m/day at top water level	13%	None as non-impounding and can turn off pumps	No pumps considered
United Utilities	1m/day	7.7%	None	Additional pumping capacity considered in contingency plans
Canal & River Trust (CRT)	Lower reservoir to 50% volume in: Category A1 – 5 days Category A2 – 7 days Category B, C, D – 9 days (note 3)	4.1%	Average winter daily flow from the reservoir catchment (except where by-wash channels or inflow diversion arrangements can reduce the flow)	Framework contractor to supply any pumping equipment. Paper by Brown (2009) says may be augmented with up to 1m^3/s by temporary pumps
Bristol Water	No standard but using 1m/day for new planned reservoir	6.5% Based on Cheddar 2 (15.5m high)	No standard; suggest average annual inflow	Only permanent can be considered reliable
Severn Trent Water	Most conservative out of (i) Hinks' formula, (ii) drawdown to 50% loading in 14 days for category A/B and 30 days for Category C/D	-	Q_{10} for Hinks	Some consideration
Welsh Water	No standard. Rely on advice from inspecting engineers for individual reservoirs	-	No standard. Rely on advice from inspecting engineers for individual reservoirs	No reliance on pumps
SSE	No standard. Rely on advice from inspecting engineers for individual reservoirs	-	No standard. Rely on advice from inspecting engineers for individual reservoirs	No reliance on pumps as scour capacity is much greater

Notes:

1. The criteria may be expressed differently in the original references but this table attempts to express the criteria in similar ways to aid comparison, using the average dam height for each owner (see notes below Table 2.2).

2. Thames Water has no policy for service reservoirs but new reservoirs require a drain to empty one cell in 12 hours. At certain times of day customer demand can drop level quite rapidly. They consider no provision is possible or necessary on the few flood storage reservoirs within their portfolio.

3. CRT criteria are based on reservoirs where there is weekly surveillance; allowed times are reduced by 2 days in cases where there is twice-weekly surveillance.

2.5 Summary of existing UK systems

2.5.1 Minimum drawdown rates

The results of the literature review, evaluation of inspection reports and the responses from major UK dam owners confirm that there was previously no single accepted published guidance on the minimum required drawdown capability for reservoir safety in the UK. This was particularly highlighted by the review of a sample of 197 Section 10 inspection reports held on the Environment Agency's database, of which less than a third of them described a documented assessment of the adequacy of emergency drawdown facilities. Even fewer of the reports referenced a specific criterion for establishing adequate drawdown capacity (only 26 of the 197 reports sampled). This confirmed the need for this guidance to be developed.

Where criteria have previously been used to assess drawdown capacity, it is expressed in several different ways making direct comparison difficult. Some criteria are based on an initial drawdown rate while others are expressed as a minimum period to achieve a global reduction in capacity or height. To provide some comparison the criteria have been normalised and expressed as a percentage of dam height per day. The dam height (or more accurately water depth) is directly related to the embankment loading. Most major UK water companies do apply a consistent in-house approach to determining the required drawdown capacity but there is a wide range of standards. Because the criteria are not generally adjusted to take into account dam height, smaller reservoirs tend to have the highest relative drawdown rate in terms of %height/day. This also means that lower category dams also have a higher relative drawdown rate.

A summary of the systems currently in use are shown in Table 2.8. Note that the basis for the existing systems is generally not stated.

2.5.2 Inflows

The assumed inflows used with the UK standards in the past are shown in Table 2.8. Three of the standards make no allowance for dealing with reservoir inflows during drawdown but the remainder generally allow for 'wet day' flows such as the Q_{10} or average flows. It was evident from the review of Section 10 reports that, in the past, inflows have generally not been taken into account when assessing the adequacy of drawdown facilities.

2.5.3 Reliance on temporary pumps

There is similar divergence in approaches adopted in the past on how much drawdown capacity may be provided by temporary pumps rather than fixed installations. Defra (2006) recommended that the permanent installation should never be less than 50% of the specified capacity. This has been adopted in both Hinks (2009) and Mann et al.'s (2014) guidance. The Canal & River Trust assume that up to 1m^3/s may be provided by mobile pumps but they have a framework contract retained to provide emergency works. There seems to be inconsistency between reservoirs owners on the reliance on temporary pumps; some owners consider them while others do not consider mobile pumps reliable enough to include as part of their minimum standard.

It is interesting to also note that in many of the emergency drawdown incidents reported (CIRIA 2014) temporary pumps were used although it is not stated how effective they were and it is possible that they may not have made a significant contribution to the reservoir lowering. However, they may well have helped with public relations (e.g. to show a proactive response to the situation).

Table 2.8 Summary of UK systems

Organisation	Drawdown criteria		Assumed inflow	% reservoir height/day	Comments
	Initial rate	Global rate			
Thames Water	1m/day		Nil	13	Most are non-impounding reservoirs
United Utilities	1m/day		Nil	7.7	
UK individual (Jonathan Hinks)	300mm/day + 5H + 8,640Q_{10}/a		Q_{10}	7.7	
Canal & River Trust		Drawdown to 50% volume in 5 to 9 days depending on consequence class	Winter daily mean inflow	2.3–4.1	
Wessex Water		Drawdown to 75% height in 3 days	0.5m^3/s	3.0	
Anglian Water		Drawdown to 50% capacity in 10 days (20 days for non-impounding/small relative catchment)	Nil	2.1 (1.0)	
Northumbrian Water		Drawdown to 25% capacity in 28 days	Winter 28-day peak	1.3	
Northern Ireland Water	Minimum 0.5m/day		Nil	2.7%	
Severn Trent Water	(i) Hinks' formula	Drawdown to 75% height in: (ii) 14 days for Category A/B (iii) 30 days for Category C/D	Q_{10} for Hinks	(i) 2.7 (ii) A/B: 1.8 (iii) C/D: 0.8	
Scottish Water	Hinks' formula for first 24 hours	CRT rule but with relaxations for specific aspects	Q_{10}	Category A: 5.4–9.0 C/D: 1.35	

2.6 Summary of international standards

2.6.1 Minimum drawdown rates

Table 2.9 shows the principal international standards identified from the literature review.

Table 2.9 Summary of international standards

Organisation	Origin	Drawdown criteria	Assumed inflow	% reservoir height/day
State of California (Babbit & Mraz 1999)	USA	**For reservoirs <6.2Mm^3 : 50% of reservoir capacity in less than 7 days.** **For larger reservoirs 10% of reservoir depth in 7 to 10 days.** (Logic appears to be that larger dams are more thoroughly designed and constructed). Excludes releases through power plants.	Nil (it is stated that in California this is true 9 months of the year)	50%: 2.94 10%: 0.35–0.5
French practice (Combelles 1985)	France	Bottom outlets should be capable of reducing load on dam by 50% in 8 days. This approximates to a dam with a storage capacity of N x 10^6m^3 requiring a bottom outlet capacity of N m^3/s.	Nil	2.6
US Bureau of Reclamation (USBR 1990)	USA	Varies with class of hazard and risk (9 Classes in Table 4).	Highest mean monthly inflows for the duration of the evacuation period	
Bureau of Indian Standards (Bureau of Indian Standards, 2004	India	Varies with class of hazard and risk: **20–50 days for 25% lowering, 40–70 days for 50% lowering and 80–100 days for 75% lowering. Overall requirement to drawdown the reservoir within a period of 1 to 4 months.**	Highest consecutive mean monthly inflows for the duration of the evacuation period	0.46–0.52
Norwegian Dam Safety Regulations (FAO 2009)	Norway	Highest class: 1m/day Second highest class: 0.5–1m/day	Average inflow	

Table 2.9 shows that, as in the UK, the international standards vary in the drawdown requirements, with values ranging between 0.46 and 2.9% height/day. While this range is less than that of the UK and towards the lower end this may be due to the standards being applicable to larger reservoirs. All of the international standards found in the literature review expressed drawdown rate as a percentage of reservoir volume and none used an initial drawdown criterion.

Many of the reservoirs overseas have a much greater capacity than UK reservoirs and the standards may not be directly applicable to the UK where forms of construction, ground conditions and rainfall patterns are different.

2.6.2 Inflows

One-half of the international standards assume no inflow when setting the drawdown capacity. The international standards that include inflows propose similar criteria to the UK, namely highest mean monthly inflows over the duration that it takes to lower the reservoir (USBR 1990, Bureau of Indian Standards 2004). The State of California ignores inflow as this is normally a realistic assumption for the majority of the year.

2.6.3 Reliance on temporary pumps

None of the standards mention the use of temporary pumping facilities, although this is probably because dams overseas tend to be much larger and pumps would be ineffective.

3 Findings from industry consultation

3.1 Introduction

Consultation with the reservoir industry was vitally important to the development of this guidance in order to:

- better understand the current approaches being applied to assessment of drawdown capability in the UK and internationally
- research the needs and views of reservoir owners and engineers who will ultimately be affected by the guidance
- gain industry buy-in to the project in order to ensure widespread uptake of the guidance

Consultation was carried out through a variety of means including:

- The whole project was overseen by a Project Steering Group made up of representatives from the industry.
- Industry feedback was obtained via two questionnaires: a UK version and an international version.
- Articles were published in the *Dams and Reservoirs* journal seeking feedback.
- Question and answer sessions were held at the 2014 British Dam Society (BDS) conference and the 2015 Supervising Engineer's Forum.

This section summarises the principal methods of industry consultation carried out and the key findings.

3.2 Project Steering Group

The whole project was overseen by a Project Steering Group made up of representatives from industry as follows:

- Mr John Ackers (Inspecting Engineer, Black & Veatch Ltd)
- Mr David Brown (Canal & River Trust)
- Mr Barry Cotter (Dwr Cymru/Welsh Water)
- Mr Dave Hart (Environment Agency, Evidence Directorate)
- Mr Ian Hope (Severn Trent Water and BDS representative)
- Dr Andy Hughes (Inspecting Engineer, Atkins)
- Mr Glyn Hughes (Dwr Cymru/Welsh Water)
- Mr Stuart King (SSE)
- Mr Roger Lewis (Environment Agency, Reservoir Safety) (PSG Chair)
- Mr Robert Mann (Inspecting Engineer)

- Mr Bryn Philpott (Thames Water)
- Mr Craig Rockliff (Environment Agency)
- Mr Ian Scholefield (United Utilities)
- Mr Russell Stead (Environment Agency)
- Dr Andy Tan (Environment Agency, Evidence Directorate)

The Project Steering Group provided advice throughout the project. At the beginning of the project they reviewed and agreed the initial scoping and viability report. They subsequently reviewed various versions of the draft guidance, and many of the panel kindly trialled early versions of the draft guidance on their own portfolio of reservoirs.

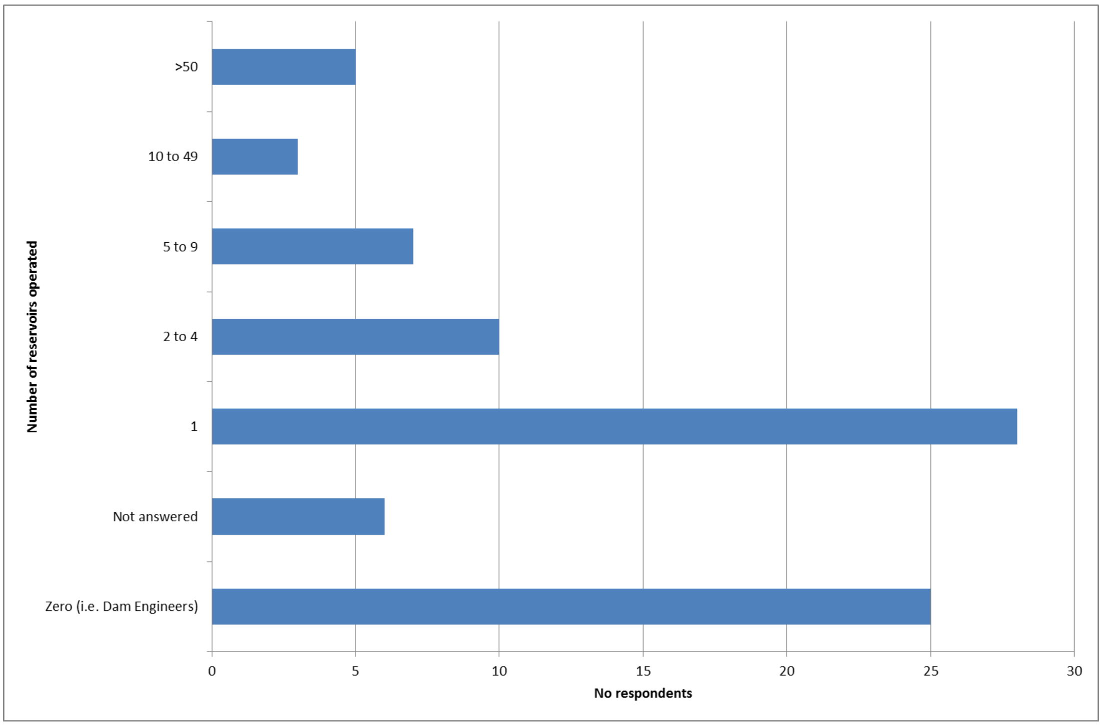
3.3 UK questionnaire

Between December 2014 and January 2015 a questionnaire was sent out to registered dam owners and all members of the BDS. A copy of the questionnaire is included in Appendix D. This section summarises the responses from the questionnaire which have been taken into account in developing the guidance.

3.3.1 Nature of respondents

A total of 84 responses were received to the questionnaire. The majority (70%) of the responses were from reservoir owners. The vast majority of these owners represented organisations and only two private owners responded. A breakdown of the respondents by the number of reservoirs operated is shown in Figure 3.1

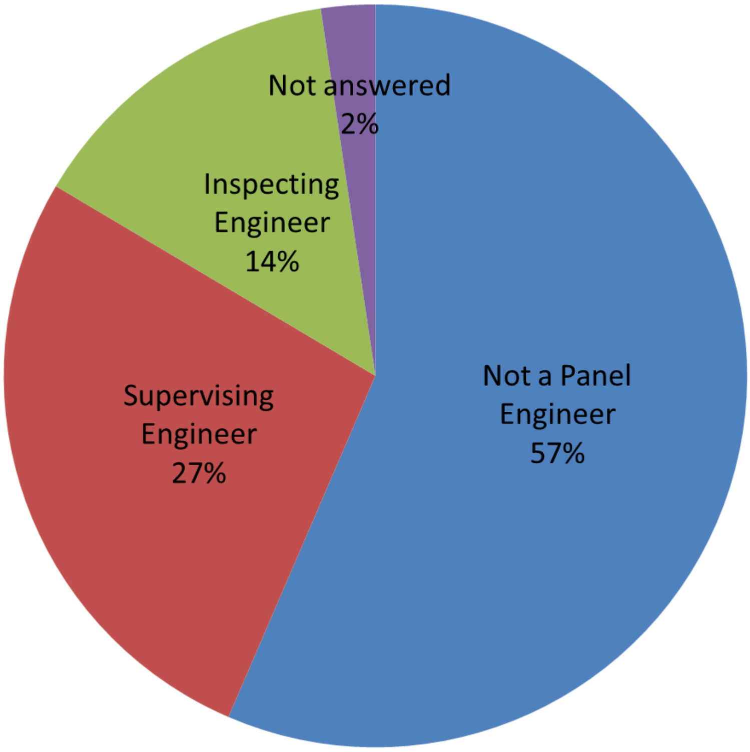
Figure 3.1 Nature of respondents to UK questionnaire (Question 1)



The owners that responded operate in total 570 reservoirs between them. The median number of reservoirs operated per owner is 1 and the maximum is 134. A small number of respondents operate most of the reservoirs in the sample (75% of the 570 reservoirs in the sample are from just five respondents representing large organisations).

Question 4 asked whether the respondents were an appointed reservoir panel engineer. The breakdown of responses is shown in Figure 3.2.

Figure 3.2 Number of reservoir panel engineers (Question 4)



3.3.2 Respondents' drawdown facilities (Q2)

The respondents representing reservoir owners were asked how many of their reservoirs had a functioning permanent emergency outlet. The results are illustrated in Figure 3.3.

Out of the 568 reservoirs operated by the respondents to this question, the large majority (90%) do have a permanent emergency outlet. However, as discussed in Section 3.2.1 the survey includes a few individuals who operate a significant proportion of the reservoir sample, and the results are therefore skewed towards the practices of larger undertakers. To account for this, the results have been normalised in Figure 3.4 to show the average split of each outlet category per respondent. On average, based on a sample of 51 reservoir portfolios, 76% of reservoirs within a typical portfolio do have a functioning emergency outlet, 22% have no outlet and 3% have a non-functioning outlet.

Figure 3.3 Presence of emergency outlets at respondents' reservoirs

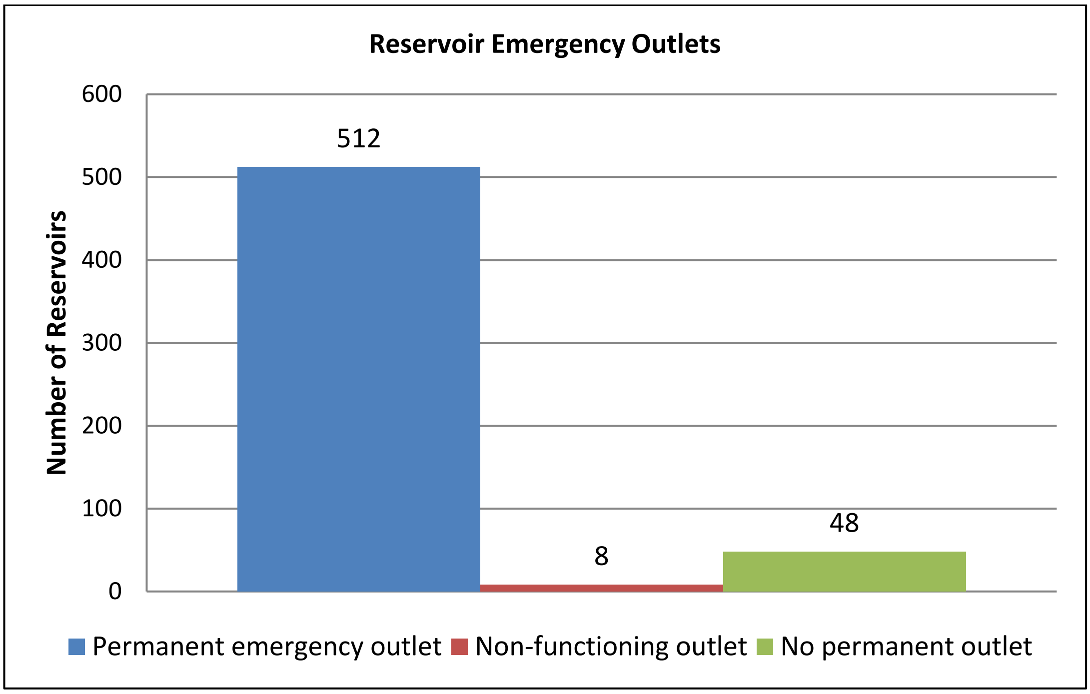
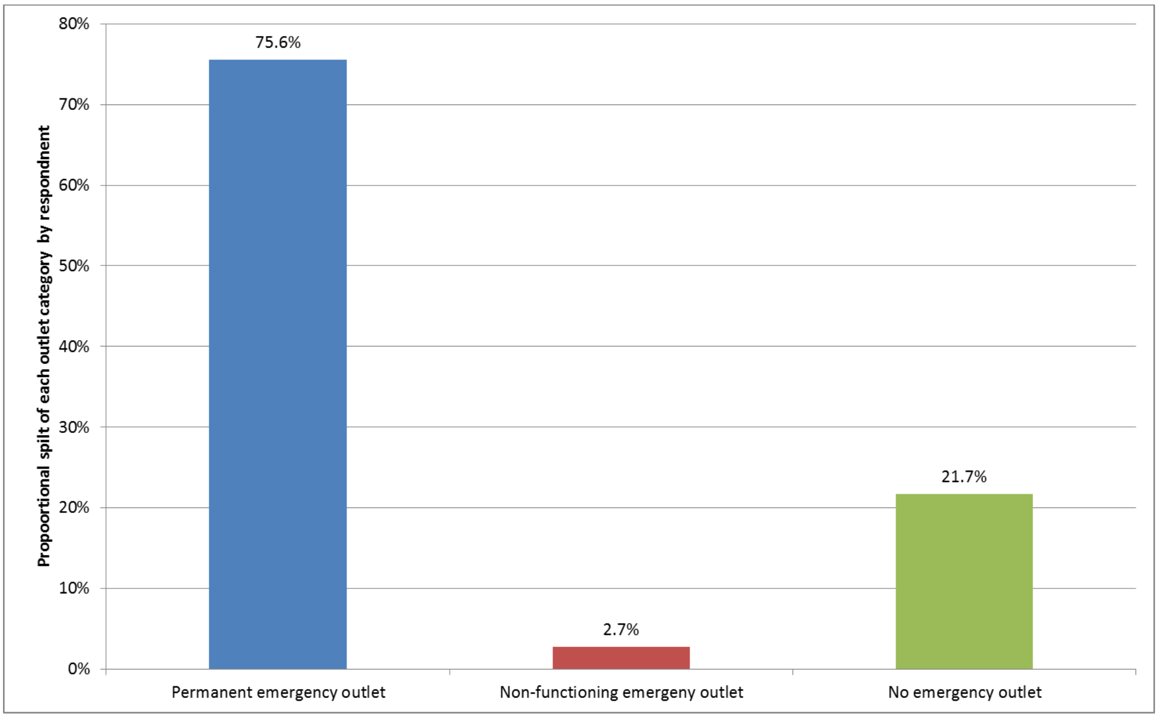


Figure 3.4 Normalised number of emergency outlets

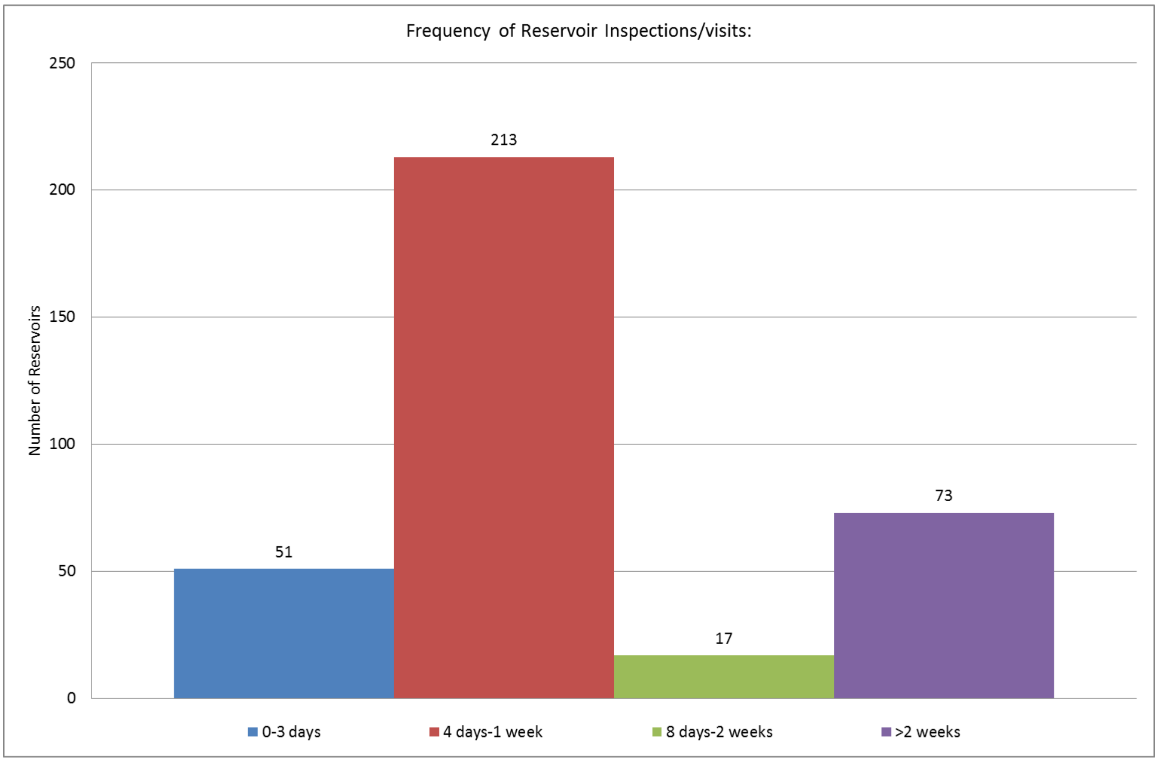


3.3.3 Surveillance practices (Q3)

Question 3 asked respondents representing reservoir owners to state the frequency of surveillance visits carried out at their reservoirs. The results are summarised in Figure 3.5. It should be noted that the graph covers 354 reservoirs, which is less than the total sample of 570 reservoirs identified by Question 1. A principal reason for this difference is that some large owners answered this question in general terms without quoting actual reservoir numbers and therefore these responses are not included on the graph. For example:

- Welsh Water, who operate 86 reservoirs, simply stated that their policy is to visit embankment dams every 4 days, concrete dams every 10 days and service reservoirs monthly.
- Yorkshire Water, who operate 134 reservoirs, simply stated that their policy is to visit embankment dams at least every 3 days and concrete service reservoirs at least weekly.

Figure 3.5 Frequency at which reservoirs are routinely visited by a responsible person



The graph in Figure 3.5 indicates that 14% of reservoirs are visited at least every 3 days and 74% are visited at least once a week. However, if the responses from Welsh Water and Yorkshire Water were taken into account, these percentages would increase.

Again, these figures are skewed towards the policies adopted by larger reservoir owners as a handful of respondents dominated the sample of reservoirs. Each response was therefore normalised, to gain a more representative idea of the visit frequency adopted by different owners. The normalised results are shown in Figure 3.6.

Figure 3.6 Normalised frequency of reservoir visits

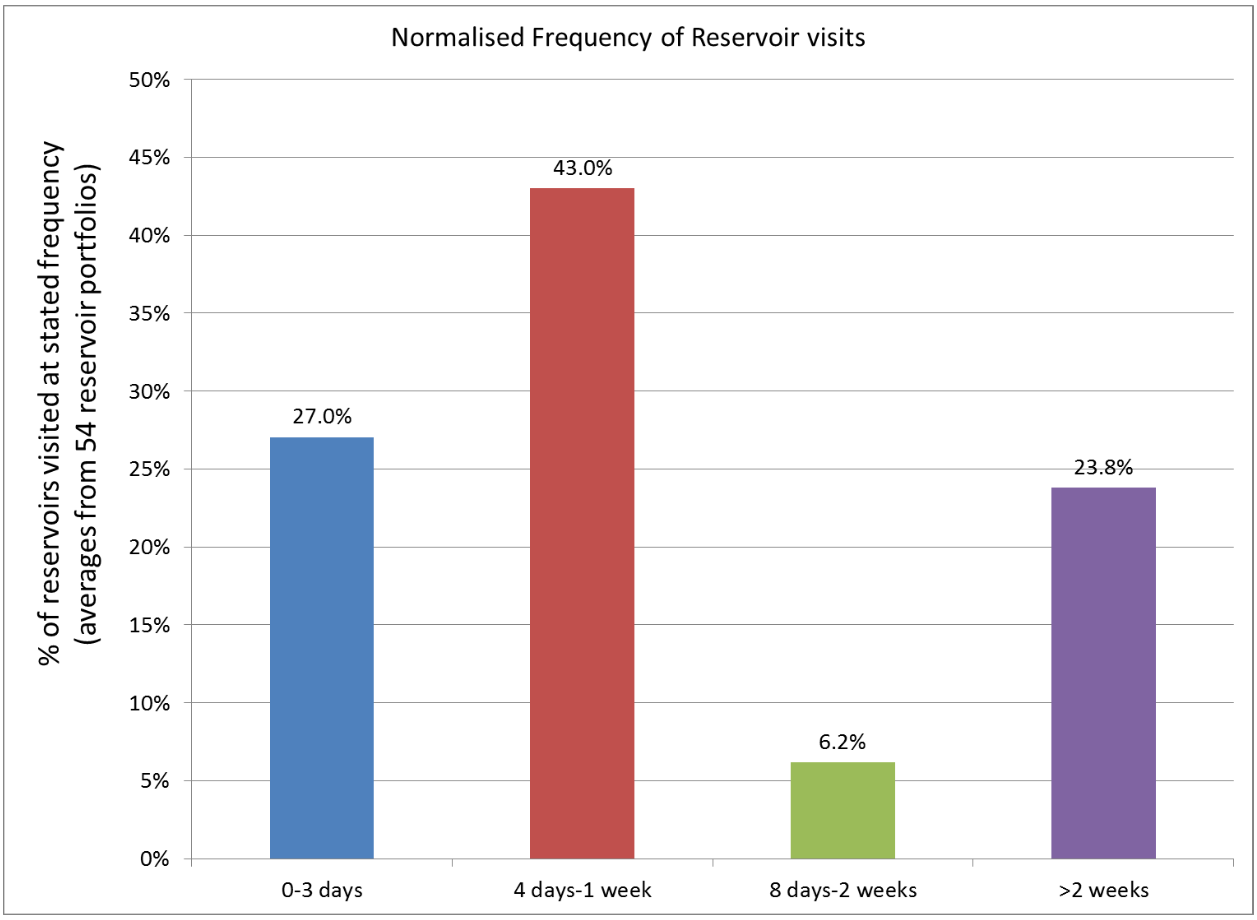
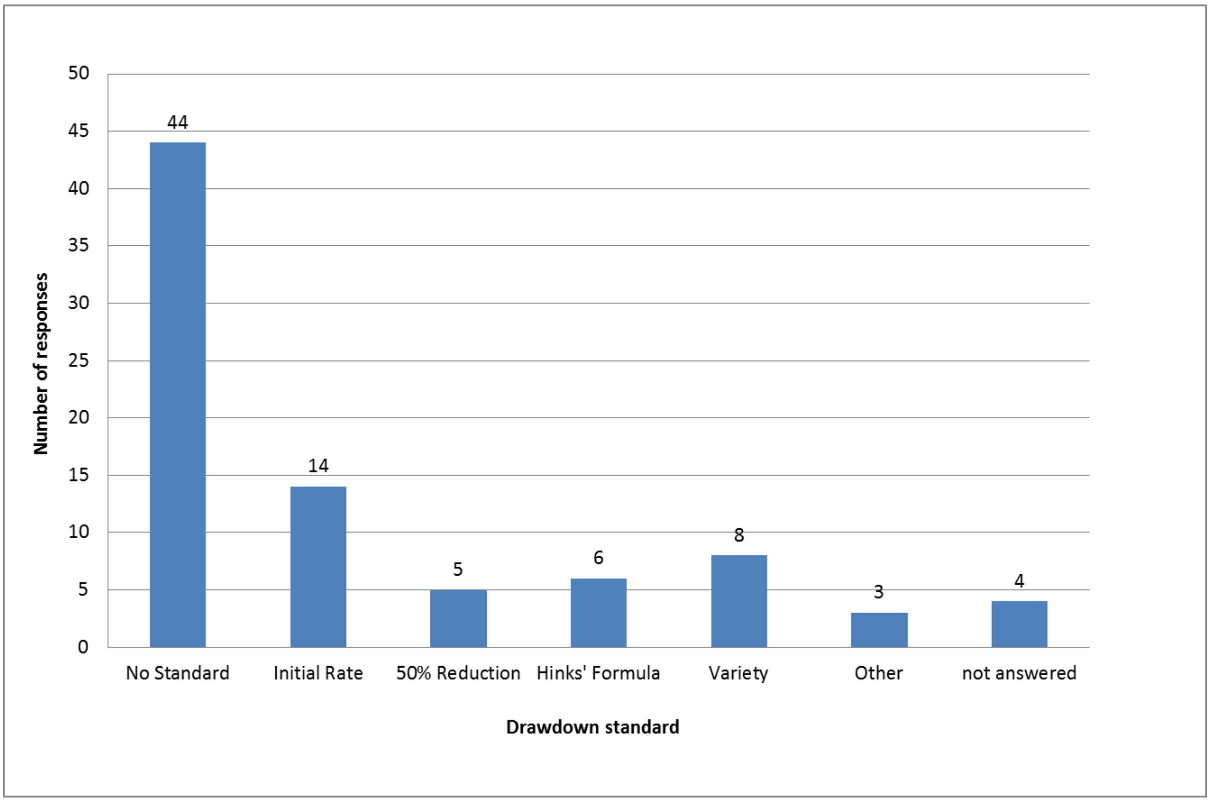


Figure 3.6 indicates that standard practice for a typical reservoir portfolio is to visit reservoirs at least once a week (70% of reservoirs in a typical portfolio are visited at this frequency) but 24% of reservoirs in a typical portfolio are visited less than fortnightly.

3.3.4 Current drawdown standards adopted (Q5)

Question 5 asked respondents whether they applied a particular rule or standard to determine the adequacy of drawdown capacity at the reservoirs they own or inspect/supervise. Just over one-half of the respondents (52%) stated that they do not have a standard. The most commonly used method was to specify an initial rate, with 17% adopting this approach. The rates adopted varied between 100mm/day and 1m/day with the average being around 0.5m/day. A variety of methods are stated to be used by 12% of respondents depending on the nature of the dam, reservoir and downstream context. Figure 3.7 shows a full breakdown of the results.

Figure 3.7 Frequency of use of different methods for determining drawdown rate



3.3.5 Allowance for reservoir inflow (Q6)

Of those who specified that they do use a standard for assessing drawdown capacity in Question 5, 73% of them use a method that accounts for reservoir inflows. The most common allowance is for Q_{10} inflows (i.e. the flow which is exceeded on average for 10% of days in a year), with 21% adopting this as an inflow allowance. A total of 24% stated that they account for inflows, but did not indicate a specific approach. The results are presented in Figure 3.8.

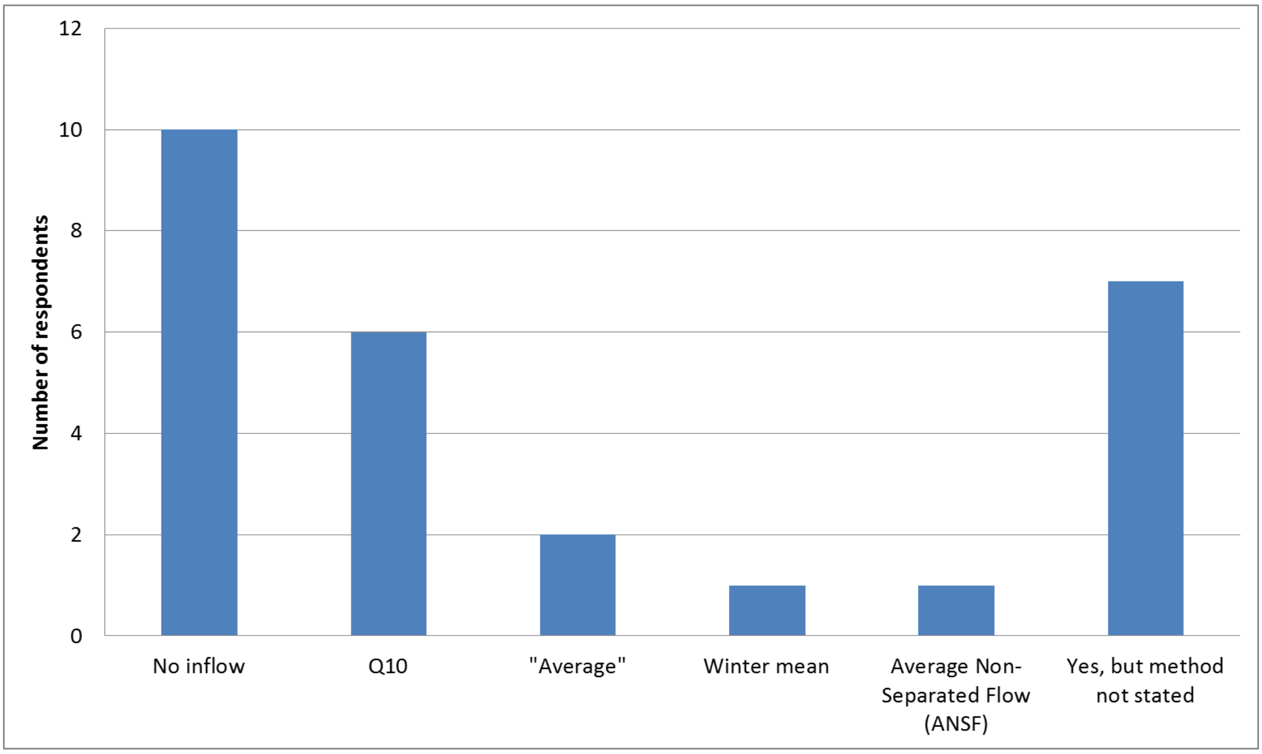
3.3.6 Necessity of permanent emergency low-level outlet (Q7)

A majority of respondents, 55%, believed that not all reservoirs require a permanent emergency outlet. The circumstances commonly referred to where such outlets are not necessary include those where:

- there are low consequences of a breach
- installation costs outweigh the benefits
- emergency pumps can be mobilised sufficiently quickly
- reservoir inflows are controlled
- there are health and safety risks associated with installation

A significant proportion of respondents, 30%, stated that all reservoirs should have a permanent emergency outlet, while 15% did not answer.

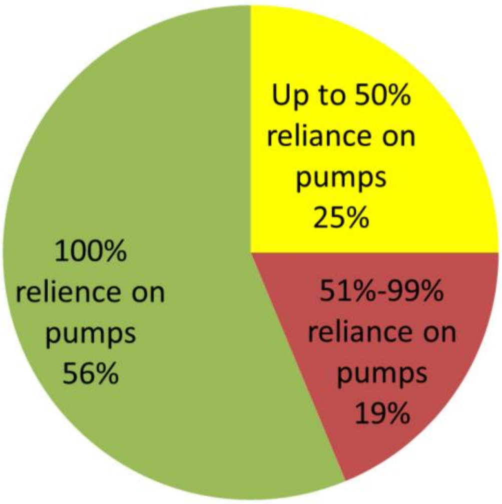
Figure 3.8 Adopted values for reservoir inflow allowance



3.3.7 Reliance on temporary pumps (Q8)

Mobilisation of pumps is required by 60% of respondents to achieve drawdown, while 40% do not require pumps. Among those who rely on temporary pumps, 16 of the respondents indicated the pump capacity they adopt as a proportion of their total drawdown capacity as shown in Figure 3.9.

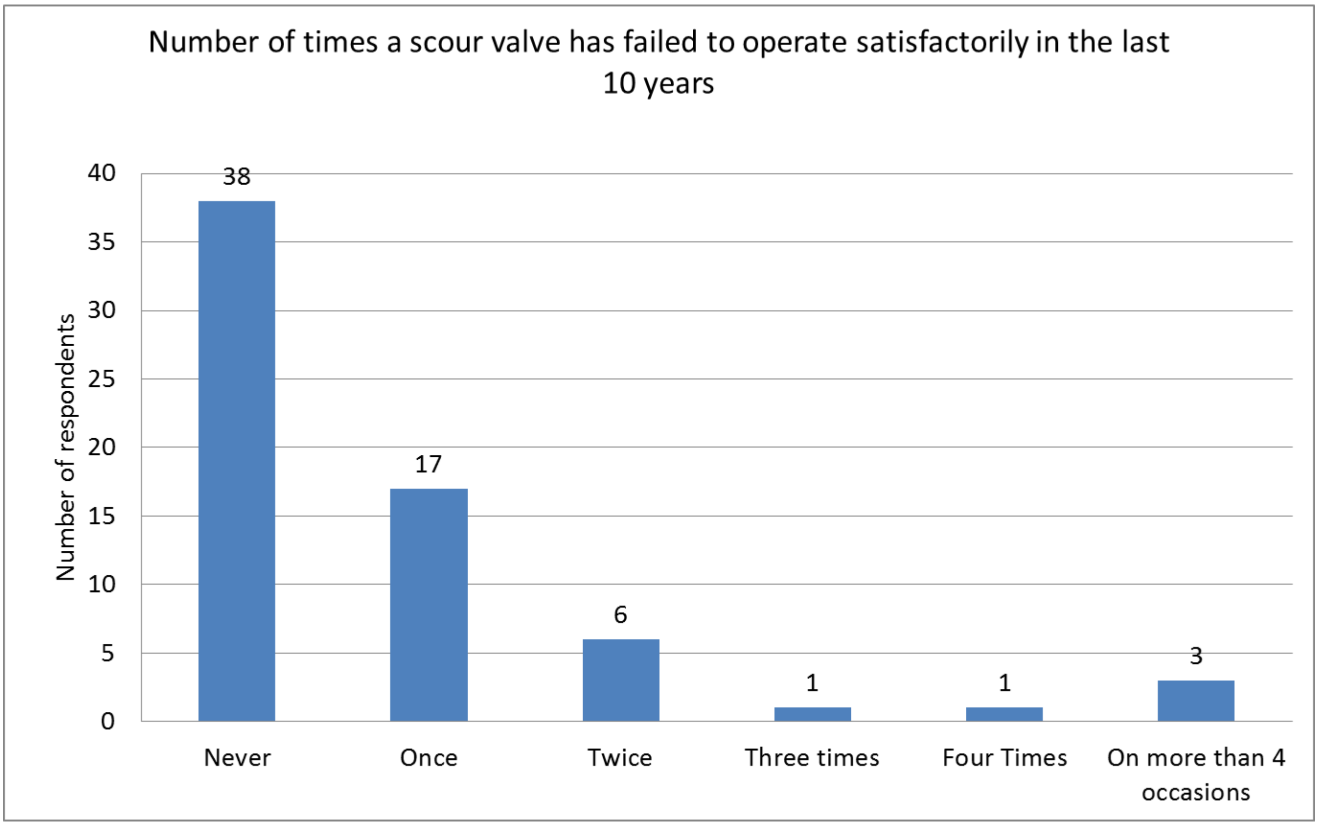
Figure 3.9 Reliance on pumps as a proportion of total drawdown capacity (based on responses from 16 respondents)



3.3.8 Number of drawdown incidents (Q9 and Q10)

A total of 36 emergency drawdown and 126 precautionary drawdown incidents were recorded by the respondents. Question 10 asked respondents to state how many times in the last 10 years a scour valve has failed to operate satisfactorily. A majority of those who answered, 57%, stated that they have not had any such incidents, 26% stated that a scour valve has failed to operate satisfactorily once in the last 10 years, while only 8% have experienced this more than twice over the same period. It should be noted that the number of scour valve failures will be significantly affected by the regularity of testing and the results may be skewed towards the practices of larger undertakers who were disproportionally represented by the survey. Figure 3.10 shows the number of scour valve failures over the last 10 years reported in the questionnaire responses.

Figure 3.10 Frequency of scour valve failures over the last 10 years



3.3.9 Lessons learned from drawdown incidents (Q11)

The questionnaire responses described a range of drawdown incidents and lessons learned, often with varying causes, severity and contexts. Similar responses were therefore grouped into categories. The number of times each category was referred to by the respondents is listed and ranked in Table 3.1.

Table 3.1 Lessons learned from drawdown incidents

Rank	Lesson learned	No. of times mentioned
1	Regularly exercise scour valves	8
2	Communicate drawdown procedures effectively	6
3 (joint ranking)	Site access difficulties during flood events	5
	Consider downstream effects of drawdown, and the need for permits	5
4	Difficulty setting up pumps during flood events	4
5	Drawdown immediately on concern being raised to allow time for further investigation	3
6 (joint ranking)	Keep entrance to valves clear	1
	Allow for malfunctioning equipment	1
	Allow for inflows (e.g. Q_{10})	1

3.3.10 Factors determining required drawdown rate (Q13)

Question 13 of the questionnaire proposed five factors which may affect the determination of an appropriate drawdown capacity and it asked respondents to rank each one from 1 (unimportant) to 5 (very important). The average rankings given to each factor are shown in Table 3.2, with the higher numbers indicating greater perceived importance.

Table 3.2 Perceived importance of factors which may affect required drawdown capacity – from UK questionnaire

Factor	**Perceived importance** (average score, where 1 is unimportant and 5 is very important)
Time to failure of the dam from first identifying a problem	4.20
Activation time (time from being informed of a problem to starting reservoir drawdown)	3.90
Potential damage downstream	3.89
Concurrent inflows during drawdown	3.45
How often a reservoir is visited	3.43

Respondents were also asked to identify any other factors that they deemed significant. Frequently mentioned factors were:

- the risk of drawdown causing downstream flooding (including during testing) (x4)
- the likelihood of failure, based on the dam's age, type and condition (x3)
- the risk of rapid drawdown causing failure of the upstream face (x2)
- the results of a cost–benefit analysis (of installing facilities to increase drawdown capacity) (x1)
- the judgement of the inspecting engineer (x1)
- the importance of the function of the reservoir; for instance, if it is the sole water supply for a large population, safety measures should be enhanced (x1)

3.3.11 Preferences for the guide (Q14)

Respondents were asked to rate the usefulness, on a scale of 1 to 5, of three different forms of guidance, which are listed in Table 3.3 along with the associated average score.

Table 3.3 Preferences for the guide

Form of guidance	Preference (average of ranking scores, where 1 is not beneficial and 5 is very beneficial)
A defined standard, or range of values, which vary depending on the dam characteristics	3.68
A quantitative method with a site-specific minimum recommended drawdown capacity	3.49
A qualitative flow chart process, describing factors that should be considered	3.46

3.3.12 Additional comments regarding a new standard (Q15)

Question 15 allowed respondents to describe any other factors or points they deem to be important in determining an appropriate drawdown capacity. The comments made most frequently are listed below:

- Private owners will often be unable to afford major re-engineering works.
- A defined standard/calculation could be impractical due to variability between reservoirs.
- Define drawdown rate in terms of water level rather than discharge.
- Simple guidance is required for smaller undertakers.

3.4 International questionnaire

In addition to the UK questionnaire described in Section 3.3, an international questionnaire was sent to a number of international contacts including major owner organisations and regulating authorities in other countries, via the International Forum of Reservoir Regulators. The questionnaires were sent out between December 2014 and January 2015 and a copy of the questionnaire is included in Appendix D. This section summarises the responses from the questionnaire which have been used in the development of the guidance.

There were 12 international respondents from the following countries:

- USA (6)
- Canada (2)
- Sweden (1)
- France (1)
- New Zealand (1)
- Austria (1)

3.4.1 Existing requirements for low-level outlets/drawdown rates (Q1)

None of the respondents stated that there is a legal requirement in their country to have an emergency/scour outlet; however, 60% stated that it is considered good practice.

3.4.2 Existing standards for drawdown rate (Q2 and Q3)

The majority of respondents, 75%, stated that their country or organisation does not have a defined drawdown standard, with many stating that the required rate is assessed on a case by case basis. Only three respondents do refer to national or company standards and of these three only one includes an allowance for inflows.

3.4.3 Factors determining required drawdown rate (Q4)

Question 4 of the questionnaire asked respondents to rank the importance given to various factors in determining the appropriate drawdown capacity for emergency purposes, using a scale from 1 (unimportant) to 5 (very important). The average rankings given to each factor are shown in Table 3.4.

Table 3.4 Perceived importance of factors which may affect required drawdown capacity – from international questionnaire (with UK responses – see Table 3.2 – in brackets)

Rank position	**Factor**	**Average score** (average score, where 1 is unimportant and 5 is very important)
1 (3)	Potential damage downstream	4.4 (3.9)
Joint 2 (1)	Time to failure of the dam from first identifying a problem	3.8 (4.2)
Joint 2 (4)	Concurrent inflows during drawdown	3.8 (3.5)
4 (2)	Activation time (time from being informed of a problem to starting reservoir drawdown)	3.2 (3.9)
5 (5)	How often a reservoir is visited	2.6 (3.4)

A potential reason why visit frequency received a lower importance rating than in the UK questionnaire responses is that many of the dams internationally are larger than those in the UK, and therefore have real-time monitoring in place. This reduces the significance of frequent visits. Other additional factors identified by the respondents include the stability of the reservoir basin, and the owner's capability/preparedness.

3.4.4 Reliance on pumps (Q5)

Most of the international respondents, 70%, do not rely on temporary pumps to achieve emergency drawdown, which contrasts with the UK responses, where 70% do rely on pumps to some extent. This may reflect the larger storage volumes of international reservoirs and the limited effectiveness of pumps on drawdown.

3.4.5 Lessons learned from drawdown exercises (Q6)

Question 6 asked respondents to describe any key lessons learned from drawdown exercises; the following lessons were described:

- scour outlet issues can be common, such as clogging/siltation
- ensure access is maintained during all conditions
- ensure downstream watercourses can accommodate additional flows

- organisation charts and instructions on-site are highly beneficial
- ensure pumps can be refuelled

3.5 Consultation with inspecting engineers

The project team delivering the project included two All Reservoirs Panel Engineers (ARPEs) and the Project Steering Group included a further three. Other ARPEs were also consulted during the preparation of the guidance as follows:

- Eight selected ARPEs and one owner reviewed a draft of the guide in autumn 2015 and completed a feedback questionnaire.
- A debate was subsequently held at the Inspecting Engineer's Forum in November 2015.

The responses from the feedback questionnaire are summarised in Figure 3.11, which highlights the mixed views among the profession.

The subsequent debate at the Inspecting Engineer's Forum consultation was focused on the following key issues:

- What is the appropriate inflow allowance?
- What is a suitable measure for expressing drawdown capacity?
- Is the erodibility of the dam material important in deciding the outlet capacity?
- Should the published document provide specific targets for drawdown capacity?

The following agreements were made at the Inspecting Engineer's Forum:

i. The recommended inflow allowance should be changed from Q_{10} to Q_{50} but it should recommend sensitivity checks to consider higher inflows.

ii. The measure for expressing drawdown should generally remain as %height/day but that it should be capped (e.g. for higher dams) in terms of a metre/day value.

iii. Erodibility of dam material should remain a fundamental consideration but the guidance should not be 'hardwired' to theoretical calculations.

iv. One further attempt should be made to draft guidance with numerical targets for drawdown capacity. If there is failure to gain agreement on the target values with the Project Steering Group then all numerical targets should be removed and instead the guidance should just describe the issues to consider.

Figure 3.11 Summary of responses from ARPE feedback questionnaire in autumn 2015

Question		ARPE 1	ARPE 2	ARPE 3	ARPE 4	ARPE 5	ARPE 6	ARPE 7	ARPE 8	Owner 1
1	Are you content that the guide does not provide specific target values for drawdown capacity?	Disagree	Agree	Strongly disagree	Disagree	Disagree	Agree	Strongly disagree	Agree	Strongly agree
2	Do you agree that the primary means of expressing drawdown rate should be as a depth (or % dam height) in the first 24 hours, rather than over a longer period?	Strongly disagree	Disagree	Disagree	No response	Agree	Strongly disagree	Strongly agree	Strongly agree	Neutral
3	Would you prefer the approach of expressing drawdown capacity in terms of the time it would take to reduce the load on the dam by say 50%?	Agree	Strongly disagree	Agree	No response	No response	Disagree	Disagree	No response	Neutral
4a	Do you agree with the proposal to adopt an inflow allowance equal to the Q_{10} flow?	Neutral	Agree	Agree	Disagree	Strongly agree	Disagree	Agree	Strongly agree	Strongly agree
4b	Would you adopt a different value for inflow allowance if the drawdown capacity was expressed in terms of the time it would take to reduce the load on the dam by 50%?	Neutral	Strongly disagree	Strongly disagree	No response	No response	No response	No response	No response	Strongly disagree
5	Would you find it useful if the guide included examples of real drawdown incidents with details of the inflows experienced, the drawdown rates achieved, and how successful the intervention was?	No response	Strongly agree	Agree	Neutral	No response	Strongly disagree	Strongly agree	Disagree	Strongly agree
6	Do you agree with the proposed title of the document?	Strongly disagree	Agree	Strongly agree	No response	No response	Neutral	Neutral	Agree	No response
7	How important is the erodibility of the dam materials, under overtopping or piping failure modes, in deciding the outlet capacity required?	Strongly agree	Strongly agree	No response	No response	No response	Agree	Agree	Strongly agree	Agree
8	Do you agree with the statement of 'the function of drawdown facilities' set out in Section 1.2?	Disagree	Strongly agree	Agree	No response	No response	Strongly agree	Agree	Strongly agree	Strongly agree

Key

Red	Strongly disagree
Orange	Disagree
Yellow	Neutral
Light green	Agree
Dark green	Strongly agree
White	No response

4 Framework to define drawdown capacity

4.1 Introduction and terminology

This section draws on the review of current practice and explains how a framework by which to assess drawdown capability was developed at the scoping and viability stage of the project. It is followed by Section 5 which provides detailed consideration of factors governing various modes of dam failure. In some cases the approach adopted in the final version of the guide was refined following the studies reported here and where this is the case it is noted in the relevant section.

This section is based on embankment dams which cover the majority of dams in the UK. Concrete dams and service reservoirs are discussed in Sections 6 and 7.

An important consideration is the terminology to be used in the guide, with the proposed definitions shown in Table 4.1. In much the same way as the *Guide to floods and reservoir safety* (ICE 1996) defines the various components of reservoir freeboard, it is useful here to define a few terms which are proposed to be adopted for the sizing of reservoir drawdown facilities.

Table 4.1 Proposed terminology in guidance on required drawdown capacity

Term	Definition/comment
Reservoir lowering capacity	This is the ability to discharge stored reservoir water only and does not include any allowance for concurrent reservoir inflows.
Pass-through capacity	This is the capacity of a bottom outlet to discharge reservoir inflows which would otherwise replenish stored water.
Drawdown capacity	This is the combined capacity of the above two components (i.e. the capacity to lower a reservoir while there is a concurrent defined inflow into the reservoir).
Risk based	The cost of increasing drawdown capacity is compared to the reduction in risk to life achieved, and only implemented where the cost is proportionate, being evaluated using values to save a life as published by the Health and Safety Executive (HSE, 2001).
Operational drawdown capacity	Capacity used to remove water for use (e.g. in water supply). Only to be taken into account in assessing reservoir safety drawdown capacity if it is reliably available in an emergency situation.

4.2 System definition

Any system to define drawdown capacity should be firmly grounded in a good appreciation of the engineering behaviour of the dam and how a drawdown facility would be beneficial. It is noted that this is not generally the case for the existing systems currently being used. It is suggested that an appreciation of the engineering behaviour is best achieved by consideration of the potential failure modes of the subject dam using the event tree defined in Figure 8.11 (applicable to embankment dams) of the *Guide to risk assessment for reservoir safety management* (Environment Agency 2013) and summarised in Table 4.2. In addition it is helpful to consider the range of possible uses of a drawdown capability as the size of drawdown capacity will depend on what stage it would be used to avert failure.

The size of drawdown capacity required will depend at which phase it is intended to be mobilised. For example, the capacity to avert initiation will be smaller than the capacity intended for ‘heroic intervention’ when a major structural problem has developed.

It is important to realise that the purpose of drawdown capacity is to avert failure once a structural problem has occurred or is believed to have occurred, and thus the critical feature is the rate of failure, and whether the drawdown can lower the reservoir sufficiently quickly to avert failure.

4.3 A tiered approach

Defining a system which is rapidly applicable to all types and sizes of dams would in many cases provide a different target capacity from a system where more detailed consideration can be given to the dam construction and operation, including assessing site-specific construction details and materials. This can be accommodated by a tiered approach where an initial rapid assessment is made which can be refined subsequently.

At the scoping and viability stage of the project, the high-level factors considered appropriate to include in the system (both tiers) were identified and include those listed in Table 4.3, with a possible matrix of these factors in Table 4.4. During the development of the guide in the later stages of the project, the two-tier system was refined. However, the principles were maintained to ensure the approach allowed for an initial rapid assessment, as well as more comprehensive, site-specific evaluations.

Tier 1 was originally proposed to be a quick screening method based on a typical reference dam(s) which would be broadly applicable to most situations. Tier 1 was intended to be comparable to the rapid method of flood estimation in *Floods and reservoir safety* (ICE 2014). Tier 2 was proposed to be a more refined method for use when:

- the situation differs from the typical reference dam(s) assumed for the Tier 1 approach (i.e. the dam is more (or less) vulnerable to threats)
- a potential deficiency has been identified and more detailed consideration is required
- the answer given by Tier 1 is considered inappropriate for any other reason
- a risk-based approach is required to evaluate the proportional cost of risk reduction

It was proposed that the Tier 2 method would take users through a series of steps to ascertain key factors which may govern the required drawdown rate. Potential factors are discussed in Section 5. Based on the outcome of these factors, it was proposed the guide would provide a quantitative method, in the form of equations, look-up tables or graphs, to allow users to calculate a site-specific value for required drawdown capacity. As the project progressed it was realised that a quantitative approach was not appropriate but a step-by-step approach to consider the relevant factors was maintained as the basis of the guide.

As the guide was developed, taking into account feedback from the industry as summarised in Section 3, the Tier 1 approach effectively became the ‘basic recommended standard for drawdown rate’.

Table 4.2 Model of intervention phases used to define drawdown capacity

	Phase	Example of matters to be considered in relation to likelihood of internal erosion	Example of matters to be considered in relation to external erosion threats	Comments in relation to drawdown capacity
1	Loading			Base flow over spillway.
2	Location of initiation			Different parameters would apply to embankment, foundation or along a structure interface.
3	Initiation		Lowering the reservoir in advance of a forecast flood could provide flood storage and mitigate overtopping damage although forecasts are generally not reliable enough to make this practical.	Critical shear stress is a useful parameter to control onset of erosion.
4	Continuation	Assess whether filters would prevent erosion continuing		
5	Progression	Will pipe stay open, or collapse? Is gradient >critical?	Lowering the reservoir could reduce wind/wave overtopping in flood recession after damage has been sustained.	Key stage in terms of defining drawdown capability. Note that internal erosion parameters are non-linear, with critical shear stress for onset of erosion, and erosion rate index once critical shear stress exceeded. Critical phase for deterioration threats where reduction of water load may reduce stress on the dam sufficiently for progression to cease.
6	Detection			Allows assessment of the impact of frequency of surveillance (and monitoring) on the required capacity. See Section 5.10
7	Intervention	Options for/effectiveness of ‘heroic intervention’		This phase is effectively the time required to mobilise/open the available drawdown capacity.
8	Breach			Once this stage is reached the reservoir has failed and the stored water is released.

Note. In reality the phases will not be as clearly defined as shown in this model and they may be in a different sequence or they may overlap. For instance, detection and intervention may occur earlier and there may be ongoing progression after the detection and intervention stages.

Table 4.3 Potential parameters to be utilised in defining target drawdown capacity

Potential parameter	Comment
Dam (water) height	It would seem reasonable for capacity to be related to dam height (i.e. non-dimensionalise). This does not necessarily fit with the concept of a critical load threshold; however, since the threshold depth is not known it is a reasonable way of demonstrating a proportionate reduction in load.
Consequences of failure	It would be proportionate to require that dams where the consequences of failure are higher have higher drawdown capability. This may alternatively be expressed as greater certainty that use of such a facility would provide effective intervention to stop failure. This would follow the principles of ALARP (i.e. to ensure that the costs of providing drawdown capacity were proportionate to the risk reduction).
Parameters describing potential vulnerability to rapid failure	These could be input as: • qualitative values (e.g. three columns for neutral, more or less vulnerable to rapid failure) • use a numeric value directly.
Frequency of surveillance	In general a drawdown capability will need to be initiated by man, and thus can only start once the incident has been detected. The time between surveillance visits is important, as this effectively prolongs the time between occurrence of a structural problem and initiation of drawdown.
Activation process for drawdown	The capacity of the outlet will also depend on how long it takes to activate. Systems that take longer to activate may need to be larger as they would be fully mobilised when the incident has had longer to develop.

Table 4.4 Illustration of possible Tier 1 matrix to determine drawdown capacity

Consequence of failure		Reservoir lowering rate expressed as % of reservoir height per day	
Option 1	**Option 2**	**Value (note 1)**	**Comments**
High A	A/B	X%	
Low A/B	C	Y%	
C/D	D	Z%	It may be that Z can be zero in certain defined situations
Notes 1. The values X, Y and Z will be determined at Stage 2 of this project. They will be precautionary/conservative values based on assessment of the time to failure, after initiation of internal erosion, for a typical reference dam(s) – see Table 4.5. 2. The drawdown capacity will need to be sized to achieve the stated drawdown rate (either X, Y or Z) plus an additional allowance to take into account concurrent reservoir inflows.			

4.4 Defining the target drawdown rate

Several ways, in principle, to determine the required drawdown capacity (i.e. to populate the Tier 1 X, Y and Z values in Table 4.4 or to develop a more refined Tier 2 method) were identified during the viability stage of the project, including:

- precedent, in terms of where drawdown capacity was effective (or ineffective) in averting failure
- expert opinion

- analysis of process, that is defined processes requiring judgement values such as the seepage and piping toolbox (USBR 2008)
- analytical calculations (system model) of rate of progression and the time to failure, after initiation of internal erosion

Methods used to derive precautionary drawdown rates are described in Sections 4.4.2 and 4.4.3 and would draw on the factors discussed in Section 5. In the case of Tier 1, the methods to be applied to a typical reference dam(s) are indicated below in order to support/derive the X, Y and Z values.

4.4.1 The Tier 1 reference dam(s)

Suggested reference dams, on which the Tier 1 method was originally proposed to be based, are shown in Table 4.5.

Table 4.5 Originally proposed reference dam(s) on which Tier 1 method was to be based

Aspect	Option 1 Typical Pennine dam	Option 2 Typical old amenity lake (heterogeneous)
Erodibility of watertight element	Sandy clay, narrow core with gravel shoulders	Very sandy clay but variable longitudinally and transversely (not zoned)
Filtering capacity	Transition zones (no rational design)	None
Typical height	15m	5m
Hydraulic gradient	5	0.2
Frequency of surveillance	Twice per week	Weekly
Activation time for drawdown (time from detection, through declaring the incident to opening the valve)	4 hours	24 hours
Freeboard	2m	0.5m
Proportion of mobile pumps	Zero	Zero

As the guide was developed, the concept of the reference dam(s) was refined with many of the parameters in Table 4.5 adopted when developing the ‘basic recommended standard’, which was based on the following specific assumptions:

- The dam is moderately susceptible to internal erosion (e.g. constructed from intermediate plasticity clay with a hydraulic gradient of around 0.2), with no designed filter. This is reasonably typical of many UK embankment dams.
- Good surveillance practices are employed, including twice-weekly visits.
- Drawdown can be activated shortly after a defect is detected.

The dam height was removed as a factor by expressing the required drawdown capacity as a proportion of the maximum depth.

4.4.2 Available precedent, expert opinion and process analysis

The following references and sources were reviewed during the project to derive recommended minimum drawdown rates based on precedent, expert opinion and process analysis:

- **Expert elicitation –** In 2004 work was carried out by KBR to survey a number of eminent dam experts about the rate of deterioration due to internal erosion. The results were processed using expert elicitation techniques and published in papers by Brown and Gosden (2004a) and Brown and Aspinall (2004).

- **Model for estimating the time for progression of piping failure** – A model was proposed by Fell et al. 2001, 2003. This forms the basis of the USBR guide known as the Internal Erosion Toolbox (USBR 2008). The model predicts likely breach times based on the soil classification and the hydraulic gradient.
- **ICOLD Bulletin 164 on internal erosion** (ICOLD 2013) – The tools developed in this bulletin were mainly focused on assessing the likelihood of internal erosion initiating rather than the rate of progression; however, it still proved to be a useful reference.
- **Existing drawdown standards used by UK dam owners –** The standards currently being used by major UK dam owners (see Section 2) were taken into account in developing the guidance and the implications of the new guidance on existing dam owners was considered.
- **Effectiveness of drawdown in actual incidents –** Actual drawdown incidents where disaster has been averted were reviewed as discussed in Section 2.2.2.

4.4.3 Analytical methods (system models)

Software exists to predict the rate of internal erosion through the progression stage and thus the time to breach. Models include BREACH, AREBA and EMBREA (see Section 8.3.5), each being developed to address weaknesses in the previous generation. The software tends to be fairly simplified, for example by considering the rate of erosion in a defined diameter hole. Much of the software is still to some extent developmental and was not commercially available at the time of this study. Consideration was given to commissioning the relevant research establishments (e.g. HR Wallingford) to run their models to assist in the project but was considered to offer limited value.

However, results from published sensitivity studies were reviewed to examine the sensitivity of output to defined ranges of input parameters. One such sensitivity was published in Section 2 of the Environment Agency's research on risk assessment methodology for small reservoirs (Environment Agency 2014).

4.5 Risk-based approach to drawdown capacity

There are now two methods of approach to reservoir safety:

- A 'standards type' approach where the design standard is arrived at based on good practice and is based on a broad categorisation of downstream damage, including the potential to endanger life, for example as detailed in Table 2.1 of *Floods and reservoir safety*, 4th edition (ICE 2014).
- A 'risk type' based approach where the risk of failure of the dam is assessed together with downstream damage, including likely loss of life, and the tolerability of that risk evaluated to arrive at the required level of protection, for example as described in the *Guide to risk assessment for reservoir safety management* (RARS; Environment Agency 2013), and the UK Health and Safety Executive's *Reducing risks, protecting people* (R2P2; HSE 2001).

The key difference is acceptance criteria. A standards approach sets out 'accepted good practice', usually with an explicit statement regarding an absolute requirement to correct.

In the risk-based approach, a dam is assessed with regard to reducing the risk of failure by increasing the drawdown capacity to 'as low as reasonably practicable' (ALARP). The ALARP principle is met when it is deemed grossly disproportionate in terms of expending resources to gain any further reduction in risk.

The approach agreed for this project was to define a ‘standards-based’ approach and categorisation but where an existing reservoir fails to meet these standards it is recommended that an engineer carries out a ‘risk-based’ assessment to review the benefit that would be gained to risk to life when compared to the costs incurred in meeting the ‘standards’ in this guide. Once more experience in the use of the risk assessment approach has been achieved a further revision of this guide may occur. For new reservoirs it is expected that a ‘standards-based’ approach will normally be adopted as the incremental cost of installing greater drawdown capacity is normally relatively small.

4.6 Summary and discussion

In summary it was agreed during the scoping and viability stage of the project that the guide would be linked to a simplified eight-step model for the typical phases of dam failure. This will promote clarity of thinking on when and how the drawdown capability would be beneficial. It was originally proposed that the guide would include a two-tier system for assessing drawdown capacity. It was anticipated that for the majority of cases only the Tier 1 approach would be needed. This would be a simplified approach based on the reservoir dam flood hazard category with drawdown expressed as a daily percentage of the dam height and is now referred to as the ‘basic recommended standard’. The reason for keeping it relatively simple is to encourage its widespread uptake within the reservoir community and also for screening purposes at Section 10 inspections under the Reservoirs Act 1975.

It was agreed in the scoping stage of the project that a Tier 2 assessment would allow users of the guide to refine the assessment (e.g. for sites which differ from the typical reference dams assumed for Tier 1). It was anticipated that the Tier 2 approach would take users through a series of steps to ascertain any factors which may govern the required drawdown rate. These factors are discussed in Section 5 of this volume. Although the approach developed in the final version of the guide has moved away from the Tier 1 and Tier 2 terminology, similar principles have been adopted.

The approach agreed for this project was to define a ‘standards-based’ approach and categorisation but where an existing reservoir fails to meet these standards it is recommended that an engineer carries out a ‘risk-based’ assessment to review the benefit that would be gained to risk to life when compared to the costs incurred in meeting the ‘standards’ in this guide. This recognises that the proposed system is to be retrospectively applied to an existing stock of dams, and is consistent with the approach as advocated in the 4th edition of *Floods and reservoir safety* (ICE 2015).

5 Factors governing drawdown capacity for embankment dams

5.1 Introduction

This section identifies the threats which could ultimately lead to structural problems with a dam and justify the provision of emergency drawdown. Threats are considered rather than the failure mode or breach mechanism because they provide a consistent starting point for identifying factors relevant to determining drawdown capacity. The list of threats has been taken from the list identified in Table 7.2 of the *Guide to risk assessment for reservoir safety management* (RARS; Environment Agency 2013).

For each threat, the following have been assessed:

- the factors that control the time to failure due to that threat (e.g. dam geometry, material types)
- the quantifiable parameters that can be used to represent each factor

The various parameters have then been scored and screened to identify those which are most relevant to the time to failure and those which are most easily ascertained (see Section 5.2). At the outset of the project it was anticipated that the guide would provide a quantitative method, in the form of equations, look-up tables or graphs, to allow users to calculate a site-specific value for required drawdown capacity. Thus it was proposed that the parameters which score highest in this screening exercise would be adopted in the calculation method. As the project progressed it was realised that such a quantitative approach was not appropriate but the selected parameters have still, by and large, been taken into account in the guidance.

As a general rule, it is not the function of the guide to assess whether a particular dam meets a defined standard to resist loading from the given threat (if such a standard exists), but once the dam has been affected by that threat it considers the time to failure and the required drawdown rate to avert failure. This corresponds to stages 3 onwards of the event tree model proposed in Table 4.2 (i.e. initiation of the threat through to breach).

5.2 Method of screening parameters

For each factor, parameters to influence drawdown capacity were selected for assessment based on judgement, and then scored by the project team, with the scoring reviewed by the Project Steering Group. At the time the scoring was carried out, the project team included three All Reservoirs Panel Engineers, of which one is a Registered Ground Engineering Adviser on the Institution of Civil Engineers UK Register of Ground Engineering Professionals (RoGEP).

Prioritisation uses a scoring system of 0 to 5, where 0 is no effect on the time to failure, and 5 is a significant effect. The candidate parameters have been scored in relation to their relevance to ‘time to breach’ and the ease of ascertaining the parameter for a particular dam. In addition, for each candidate parameter, an assessment has been made regarding whether there is a viable method of linking it to the time to failure. For instance, a parameter may theoretically be relevant to the time to failure but if there is no practical method to quantitatively relate it (e.g. published empirical relationships) then it was not considered viable for use in the guide.

5.3 Floods

5.3.1 Introduction

This subsection assesses the potential parameters which were considered in determining the effect that the threat from floods has on the required drawdown capacity.

5.3.2 Potential damage and failure modes from floods

Table 7.2 of RARS provides a summary of the typical failure modes arising from floods for embankment dams. These may be grouped under three broad headings:

- structural failure of spillway: from high flow velocity
- structural failure of embankment: from overflowing of crest or the spillway channel, saturating and/or eroding the fill leading to slope instability
- internal erosion (four types as in ICOLD Bulletin 164 (ICOLD 2013)) from increased hydraulic loading

The direct risk of failure from a flood (i.e. overflowing and scour of the downstream face and backcutting through which the contents of the reservoir can then flow) is normally dealt with through the provision of a spillway, with the size of the flood which the dam must be capable of withstanding being based on the potential downstream hazard resulting from failure under flood conditions.

While drawdown is unlikely to be able to directly reduce the risk of failure from a flood, it could play a part in preventing the subsequent release of water as failure modes develop as a result of damage from a flood event. Such damage could result from either the design flood/loading having been exceeded or a latent defect in the dam being triggered by the increased hydraulic loading.

5.3.3 How can drawdown capability avoid the risk of failure from floods

Table 5.1 shows ways in which drawdown can be used at different stages to help avoid flood risk.

Table 5.1 Drawdown and the avoidance of flood risk

Stages	Potential for drawdown to be useful
Prior to flood event occurring	In principle lowering the reservoir level in advance of a flood event would be beneficial, as it would reduce the subsequent flood rise/hydraulic loading and overflow discharge. Typical ratios of catchment areas to storage volume in the UK suggest that the effect of doing so is likely to be limited for the majority of UK reservoirs.
During a flood event	For most reservoirs the inflow rate and flood volume are many times larger than the discharge that can be economically provided by a low-level outlet. Under most circumstances drawdown during a flood event would have limited effect on reservoir level and spillway discharges until the flood is in recession.
Following a flood event	If following a flood event a dam is seen to be damaged, damage is continuous with base flow, or unseen damage is thought to have occurred, lowering and maintaining the reservoir at a lowered level has the potential to prevent catastrophic release of the reservoir contents.

5.3.4 Potential damage and failure modes

Table 5.2 lists dam failure modes due to floods.

Table 5.2 Embankment dam failure modes due to floods

Mode	Damage (Initiation)	Progression and subsequent failure mode	Parameters which will influence risk of breach
1	Structural failure of spillway chute resulting in erosion of embankment toe/fill (e.g. Boltby in 2005 and Ulley in 2007)	Downstream slope failure resulting in loss of freeboard (i.e. settlement of crest) then overflowing and erosion of downstream face and backcutting through which the contents of the reservoir can then flow;	Erodibility of downstream slope material Downstream slope profile/material strength Width of dam crest/freeboard
		Potential for progressive downstream slope failure from subsequent flood events if spillway chute overflows; and/or	Size of low-level outlet relative to catchment area to prevent subsequent discharge
		Downstream slope failure increasing hydraulic gradient across core and initiation of internal erosion (for further progression details see below).	See mode 2
2	Overflowing of dam crest erodes downstream face/toe, or saturates downstream fill	Downstream slope failure resulting in loss of freeboard then overflowing and erosion of downstream face and backcutting through which the contents of the reservoir can then flow.	Erodibility of downstream slope material Downstream slope profile/material shear strength Width of dam crest/freeboard
3	Spillway chute capacity is exceeded resulting in out of channel flow eroding embankment toe/fill	Downstream slope failure resulting in loss of freeboard then overflowing and erosion of downstream face and backcutting through which the contents of the reservoir can then flow; and/or	Erodibility of downstream slope material Downstream slope profile/material shear strength Width of dam crest/freeboard
		Downstream slope failure increasing hydraulic gradient across core and initiation of internal erosion (for further progression details see below).	See mode 4
4	Elevated water level (increased hydraulic gradient): (i) initiates internal erosion (concentrated leaks, backward, contact, suffusion), or; (ii) saturates downstream shoulder	(i) Pipe enlarges and collapses, or loss of fine material due to suffusion continues, resulting in loss of freeboard then overflowing and scour of downstream face and backcutting through which the contents of the reservoir can then flow. (ii) Slope failure (from saturated fill) resulting in loss of freeboard then overflowing and erosion of downstream face and backcutting through which the contents of the reservoir can then flow.	Downstream slope profile/material strength Width of dam crest Erodibility of core material Filtering capability of filter and shoulder

Note: Modes 2, 3 and 4 are modes that occur at the peak of the flood and are only relevant to drawdown if sufficient damage is caused at the peak to cause failure at more normal water levels. Mode 1 will continue with just base flow into the reservoir.

5.3.5 Relationship between parameters that influence risk of failure and effectiveness of drawdown

Parameters which influence the vulnerability of a particular dam following damage from a flood are evaluated in Section 5.3.6. These parameters influence the time of failure due to overflowing. Parameters that influence the time of failure due to internal erosion are considered separately in Section 5.8. In both cases the progression phases are essentially the same, even though the initiating threat is different.

In terms of the threat from flooding, the impact of drawdown is either to reduce the reservoir level sufficiently to prevent further overflowing of a damaged dam or spillway, or to slow the rate of progression of internal erosion.

One of the difficulties is that all breach modelling uses the detachment coefficient k_d, which is a surface erosion parameter (see Equation 2.1 in Environment Agency 2014), and that there are a range of equations to estimate k_d from other parameters as shown in Table 5.3.

Table 5.3 Derivation of erosion coefficient, k_d (from Table 2.2 of Environment Agency 2014)

	Method	Reference to Morris PhD (Morris 2011)	Key features	Range of k_d shown in Morris 2011	Range of K_d considered reasonable for UK dams	
					Most erodible – SM	Least erodible – CH
1	Temple and Hanson (1994)	Eqn 6.5	Function of dry density and clay content		10 (dry density 1.2, 0% clay)	0.04 (dry density 1.8, 80% clay)
	Regazzoni (2009)	Eqn 6.11–6.13	Liquid limit, clay content, degree of saturation		No data to allow application to UK soils	
2	Hanson (2007)	Eqn 6.6	Compactive effort and water content		Lower values than above	
3	Qualitative description	Table 6.4		<0.001 to >20	20	0.01
4	Link to degree of compaction and clay content (Hanson)	Table 6.5, Figure 6.14		0.001 to 1,000	100	0.01
5	Direct measurement Jet erosion test Hole erosion test		The two tests seem to give different results			

Notes.

1. The terms SM and CH refer to the unified soil classification system with SM being a silty sand and CH being a high plasticity clay.

5.3.6 Screening of flood factors

The parameters were scored as described in Section 5.2. A total of five candidate factors have been identified in Table 5.4, with the overall score varying between 6 and 9. Factors with a score of 8 or more are:

- crest width
- freeboard
- erodibility of soil in dam - detachment coefficient, k_d

These parameters were therefore initially proposed to form part of a qualitative system for assessing the required drawdown rate. As the guidance was developed the approach and parameters were refined; crest width was considered to be captured by the hydraulic gradient and the erosion rate index was considered a better parameter to take into account the general erodibility of dam fill. Freeboard was not specifically included in the guide except as part of the critical failure modes assessment.

Table 5.4 Screening of flood factors that influence the risk of overtopping following a flood and speed of failure

Factor/parameter		Possible system(s) to quantify		Comment	Scoring (0 – not relevant to 5 – highly relevant)			Viability of quantifying link
Type	**Description**	**Parameter**	**Criteria for fast failure**		**Relevance to 'time to breach'**	**Ease of assessment**	**Total**	
Dam geometry	Crest width	Crest width		Model using a hydraulic breach model, such as BREACH (Innovyze Infoworks RS)	3	5	8	Yes
	Freeboard from spillway to dam crest	Freeboard			4	5	9	Yes
	Downstream slope angle	Angle			3	4	7	Yes
Material properties – embankment and foundation	Fill compaction	Construction methods			3	3	6	Yes
	Vulnerability of downstream face to erosion	Includes slope angle, material type, type of surface	Low width/high flow		3	4	7	Yes
	Erosion rate, k_d				5	3	8	Yes
						Max	9	
	Number of candidate factors	5				Min	6	

5.4 Wind (waves)

5.4.1 Introduction

This subsection assesses the potential parameters which were considered in determining the effect that the threat from waves has on the required drawdown capacity.

5.4.2 Potential damage and failure modes from waves

Table 7.2 of the RARS guide (Environment Agency 2013) provides a summary of the typical failure modes arising from waves for embankment dams. These may be grouped under the broad headings of:

- Scour of embankment: from failure of upstream slope protection leading to erosion of upstream fill, loss of freeboard, overflowing and erosion of fill.
- Structural failure of embankment: from wave overtopping of crest due to inadequate freeboard or collapse of wave wall, saturating and/or eroding the fill leading to slope instability.

Drawdown is unlikely to be able to directly reduce the risk of failure from the concurrent threats of wind and floods.

It could play a part in preventing the subsequent release of water as failure modes develop as a result of damage from waves.

5.4.3 How can drawdown capability mitigate the risk of failure from waves

The potential benefits of reservoir drawdown in relation to the threat from waves are discussed in Table 5.5.

Table 5.5 Benefits of drawdown in relation to waves

Stages	Potential for drawdown to be useful
Prior to wind (waves) occurring	In principle reducing the reservoir level in advance of windy weather would be beneficial, as it would increase the freeboard and hence reduce the likelihood of wave overtopping and loading on crest walls, or erosion of the upstream face removing all freeboard. Severe winds can be predicted several days in advance but operators are unlikely to want to lose water.
Once wind has developed	A typical UK reservoir will develop waves fully in 10–20 minutes, so there would be little warning time (ICE 2015, p.26).
Following high wind (waves)	If a dam is seen to be damaged or unseen damage is thought to have occurred as a result of high waves, lowering and maintaining the reservoir at a lowered level has the potential to prevent the progression of failure modes.

5.4.4 Potential damage and failure modes

The potential damage and failure modes which could result from wind and waves are discussed in Table 5.6.

Table 5.6 Potential failure modes caused by waves

Damage (initiation)	Progression and subsequent failure mode	Parameters which will influence risk of breach (see note 1)
Large waves result in failure of erosion protection to upstream face	Erosion of upstream fill by wave action resulting in localised slope failure/lowering of crest, and overtopping (most likely during a subsequent flood event).	Initiation: • fetch length • reservoir axis • upstream protection type/size/slope Progression: • freeboard • reservoir fetch length • erodibility of upstream fill • inflow
Large waves result in overtopping of dam crest	Downstream slope failure (from saturated fill and/or erosion damage) resulting in loss of freeboard then overflowing and erosion of downstream face and backcutting, through which the contents of the reservoir can then flow.	Initiation: • fetch length • reservoir axis • upstream protection type/size/slope Progression: • freeboard • reservoir fetch length • erodibility of downstream fill • inflow

Note 1: The parameters which will influence the risk of breach have been separated into those relating to initiation and those relating to progression. The former affect the risk of damage occurring to the dam but not the ongoing deterioration to breach (i.e. a single event will probably not result in failure). Once the wind event drops, there is no further vulnerability and therefore no value in drawdown other than to keep the reservoir low to prevent related damage and to effect repairs.

5.4.5 Relationship between parameters that influence risk of failure and effectiveness of drawdown

The parameters given under the heading initiation in Table 5.6 effectively influence how vulnerable a particular dam is to damage caused by waves. The parameters given under the heading progression control the progression to failure after damage has occurred. The impact of drawdown is to reduce the reservoir level sufficiently to prevent further erosion of the upstream face, or overflowing of a damaged dam.

5.4.6 Screening of wind factors

The parameters have been scored as described in Section 5.2. A total of eight candidate factors have been identified in Table 5.7, with the overall score varying between 6 and 9. Factors with a score of 8 or more are:

- crest width
- freeboard

These parameters were therefore initially proposed to form part of a qualitative system for assessing the required drawdown rate. As the guidance was developed the approach and parameters were refined. Crest width was considered to be captured by the hydraulic gradient. Freeboard was not specifically included in the guide except as part of the critical failure modes assessment.

Table 5.7 Screening of wave factors that influence risk of overtopping and speed of failure

Factor/parameter		Possible system(s) to quantify		Comment	Scoring (0 – not relevant to 5 – highly relevant)			Viability of quantifying link
Type	**Description**	**Parameter**	**Criteria for fast failure**		**Relevance to 'time to breach'**	**Ease of assessment**	**Total**	
Reservoir geometry and inflow	Fetch length				2	4	6	Yes
	Reservoir axis				2	4	6	Yes
	Inflow	Taken into account separately						
Dam geometry	Crest width	Crest width			3	5	8	Yes
	Freeboard from spillway to dam crest	Freeboard			4	5	9	Yes
Material properties – embankment and foundation	Fill compaction	Construction methods			3	3	6	Yes
	Erodibility of upstream face	Type/size/slope of upstream protection			4	3	7	Yes
	Vulnerability of downstream face to wave overtopping				3	4	7	Yes
						Max.	9	
	Number of candidate factors	8				Min.	6	

5.5 Upstream dam

This subsection assesses the potential parameters which were considered in determining the effect that an upstream dam has on the required drawdown capacity.

It draws on previous research contracts and other technical reports including Table 8.8 of the RARS guide (Environment Agency 2013) that discuss the risk posed by upstream dams. Conclusions are summarised in Table 5.8, which suggests that detailed assessment of the risk is not normally required since it is a matter for the owner of the upstream dam.

Table 5.8 How can drawdown capability avoid the risk of failure from an upstream dam?

Risks from upstream dam	Potential for drawdown to be useful
Failure of upstream dam	While in principle it may be possible in some circumstances that a reservoir could be lowered to provide space to absorb the releases resulting from failure of an upstream dam, in many cases this will not be possible. It is the responsibility of the upstream dam owner to manage the risk of failure of his or her dam and it would be unduly onerous to impose a requirement on a downstream dam owner to provide drawdown capacity to avoid consequential failure of the upstream dam even if it were physically possible. This will not be considered further.
Discharge of floods from upstream dam	This should already have been considered in evaluating the safety of the downstream dam and its own spillway discharge facilities should take this into account.
Discharge from emergency drawdown	The discharge from an emergency drawdown outlet would normally be significantly lower than that from a spillway, so the existing spillway capacity should be adequate. There may be rare cases where this is not the case and where the spillway capacity is very small the drawdown capacity of the subject dam may need to take into account the drawdown capacity of the upstream dam.

Drawdown to limit the damage caused by failure of an upstream dam and consequential failure of the downstream dam has not been considered as a criterion for establishment of the drawdown capacity. Where dams in cascade are under the same ownership this may be something that could be considered when establishing the drawdown capacity of the whole cascade; for example, it may be more cost-effective to provide a large drawdown capacity at the lowest dam of a cascade which would allow it to absorb the breach flood from an upstream dam rather than provide drawdown capacity at each of the dams in the cascade.

The risk of an upstream reservoir requiring emergency drawdown at precisely the same time as drawdown being required at the reservoir under consideration is considered too low probability to be factored into the guidance.

5.6 Ice

5.6.1 Introduction

This subsection assesses the potential parameters which were considered in determining the effect that the threat from ice has on the required drawdown capacity.

It draws on the guidelines in ICOLD Bulletin 105 *Dams and related structures in cold climates* (ICOLD 1996). The intent of the guide appears to be focused on providing an understanding of ice problems for Arctic conditions, but the general principles are applicable to UK dams.

5.6.2 Potential damage and failure modes from ice

Ice pressures can produce a significant load against the face of a dam in locations where winter temperatures are cold enough to cause relatively thick ice cover. In the UK a thickness of 400mm is cited as being the maximum that can be envisaged for severe conditions (CIRIA 1996a).

Horizontal loads from ice are produced by the thermal expansion of the ice sheet and by wind drag. For flexible structures such as embankment dams, horizontal loading from ice is not normally a major factor in stability calculations, but thermal expansion can result in significant forces on rigid, rectilinear structures, such as concrete dams (see Section 6).

In the case of embankment dams, the sloped upstream face normally limits the effects from ice to displacement of rip rap/rock armour type protection, these being more vulnerable to displacement than relatively smoother protection such as slabs, asphaltic concrete or stone pitching, where the ice sheet will tend to ride up/down the face.

Rip rap can be damaged by the action of ice cover as it drops in response to falling water level, such as that induced by drawdown. The mechanism is as follows: as the water level drops the ice cover cracks leaving a band frozen to the rip rap, which first forms a cantilever before collapsing onto the embankment surface, subjecting frozen stones to overturning and torsional moments and moving them into new positions. When the ice thaws these stones fall but rarely return to the same position. When repeated, this process causes the gradual deterioration of the rip rap, rendering it unstable. Without maintenance, waves will erode the rip rap and potentially the dam fill.

It is reported that rip rap displacement has occurred on slopes steeper than 1V:1.75H (typical for rockfill dams) when the ice cover is thick and the reservoir level drops, but where slopes are 1V:3H (typical of that for many UK embankment dams), only minor movement has occurred.

It is therefore concluded that unlike concrete dams (discussed in Section 6), intentionally drawing down the reservoir once the ice sheet has developed could result an in increased likelihood of damage to the rip rap.

5.6.3 How can drawdown capability avoid the risk of failure from ice?

Table 5.9 lists benefits of drawdown for iced reservoirs.

Table 5.9 Benefits of drawdown for iced reservoirs

Stages	Potential for drawdown to be useful
Prior to ice sheet forming	Little perceived benefit: The potential for damage would simply be transferred to a lower level on the upstream slope.
Once an ice sheet has formed	Drawdown could increase the likelihood of damage.
Following ice sheet melting	If the protection to the upstream face is seen to be damaged, lowering the reservoir has the potential to enable repairs to be undertaken to the rip rap before wave action and erosion threatens the dam.

5.6.4 Potential damage and failure modes

Table 5.10 lists potential failure modes from ice.

Table 5.10 Potential failure modes from ice

Damage (initiation)	Progression and subsequent failure mode	Parameters which will influence risk of breach
Ice sheet results in displacement of rip rap protection to upstream face	Erosion by wave action resulting in localised slope failure/lowering of crest, and overtopping (most likely during a subsequent flood event)	Progression: • reservoir fetch length • erodibility of upstream fill freeboard • inflow

5.6.5 Summary

Ice is not considered to be a significant threat to embankment dams within the UK and to warrant specific consideration regarding drawdown provision that is not otherwise covered by the other more likely threats.

5.7 Earthquake

5.7.1 Introduction

This subsection assesses the potential parameters which were considered in determining the effect that earthquake threat has on the required drawdown capacity.

It draws on previous research contracts and other technical reports including

- RARS guide (Environment Agency 2013); other than a row in Table 8.4, earthquakes are not referenced in this document

Table 5.11 summarises the potential benefits of reservoir drawdown in relation to the threat from earthquakes.

Table 5.11 How can drawdown capability avoid the risk of failure from earthquakes?

Stages in an earthquake	Potential for drawdown to be useful
Prior to an earthquake	In principle reducing the reservoir load in advance of an earthquake would be beneficial. However, methods of earthquake prediction are not sufficiently mature to make such an approach reliable.
During an earthquake	An individual earthquake event lasts for seconds and it is not feasible to take any action within that timeframe. Aftershocks of lesser magnitude than the main event are common. These are normally of lower magnitude and are unlikely to initiate a failure that was not already underway. Any action would only be beneficial if damage was already evident or presumed and action would be taken in the same way as following an earthquake below. This is not considered further.
Following an earthquake	If following an earthquake event a dam is seen to be damaged or unseen damage is thought to have occurred, lowering the reservoir has the potential to prevent catastrophic release of the reservoir contents.

While drawdown cannot do anything to directly reduce the risk of failure from an earthquake it could play a part in preventing the subsequent release of water as failure modes develop as a result of the damage during an earthquake.

5.7.2 Relationship between parameters that influence risk of failure and effectiveness of drawdown

Table 5.12 shows parameters which effectively influence the vulnerability of a particular dam to damage following an earthquake. The impact of drawdown is either to reduce the reservoir level sufficiently to below the new crest level to prevent overtopping or slow the rate of progression of internal erosion.

The latter will be covered comprehensively in Section 5.8 and only the former will be considered here.

Table 5.12 Potential damage and failure modes following an earthquake

Damage	Subsequent failure mode	Parameters which will influence risk of breach
Settlement of embankment	Lowering of crest and overtopping	Geotechnical properties which relate to consolidation or liquefaction Freeboard Erodibility of downstream face
Settlement of foundation	Lowering of crest and overtopping	Geotechnical properties which relate to consolidation or liquefaction Freeboard Erodibility of downstream face
Slope failure	Degradation of slope, lowering of crest and overtopping	Downstream slope profile Width of dam crest
	Increase in hydraulic gradient across core and internal erosion	Erodibility of core material Filtering capability of filter and shoulder
Disruption of filters	Reduction in filtering capacity and internal erosion	Width of filters

5.7.3 Screening of earthquake parameters

The parameters have been scored in Table 5.13 as described in Section 5.2. A total of seven candidate factors have been identified, with the overall score varying between 5 and 9. Factors with a score of 8 or more are

- crest width
- freeboard

These parameters were therefore initially proposed to form part of a qualitative system for assessing required drawdown rate. As the guidance was developed the approach and parameters were refined. Crest width was considered to be captured by the hydraulic gradient. Freeboard was not specifically included in the guide except as part of the critical failure modes assessment.

Table 5.13 Screening of earthquake factors that influence risk of overtopping following earthquake and speed of failure

Factor/parameter		**Possible system(s) to quantify**		**Comment**	**Scoring (0 – not relevant to 5 – highly relevant)**			**Viability of quantifying link**
Type	**Description**	**Parameter**	**Criteria for fast failure**		**Relevance to 'time to breach'**	**Ease of assessment**	**Total**	
Dam geometry	Crest width	Crest width			3	5	8	Yes
	Freeboard from spillway to dam crest	Freeboard			4	5	9	Yes
	Downstream slope angle	Angle			3	4	7	Yes
	Disruption of thin watertight element (e.g. upstream membrane or thin core)	Presence of thin watertight element			4	2	6	Yes
Material properties – embankment and foundation	Fill compaction	Construction methods			3	3	6	Yes
	Susceptibility to liquefaction	Material type			3	2	5	Yes
	Vulnerability of downstream face to overflow				3	4	7	Yes
						Max	9	
	Number of candidate factors	7				Min	5	

5.8 Deterioration

5.8.1 Introduction

This subsection assesses potential deterioration parameters which should be considered in determining the effect of deterioration of the dam on the required drawdown capacity.

It draws on previous research contracts and other technical reports including:

- ICOLD (2013) – Bulletin 164, *Internal erosion of existing dams, levees and dikes, and their foundations*
- Environment Agency (2011) – *Modes of dam failure and monitoring and measuring techniques*
- Defra (2007) – *Engineering guide to early detection of internal erosion*
- Fell et al. (2001) – *The* t*ime for development and detectability of internal erosion and piping in embankment dams and their foundations*

5.8.2 Deterioration parameters which are most relevant to drawdown capability

The steps in progress of internal erosion from initiation to failure, as stated in ICOLD Bulletin 164 and RARS Figure 8.11 (Environment Agency 2013) are summarised in Table 4.2.

The speed of deterioration (time to failure) in Phase 5, Progression, is important because it links directly to the time available between detection and mobilising any drawdown capacity. However, it should be noted that drawdown capacity is equally (if not more) dependent on:

- Phase 6 – frequency of surveillance (and monitoring) – see Section 5.10
- Phase 7 time required to mobilise/open the available drawdown capacity – see Section 5.11

Thus the parameters which may be important in terms of speed of deterioration are likely to also be important as factors considered in surveillance and monitoring, and which are used to define when there is some concern over structural behaviour.

A further complexity is that deterioration parameters may include:

- feature absent/present (or may be present)
- one parameter for initiation or limitation of damage, and a second parameter for rate of erosion

In principle both could be used to influence drawdown capacity.

Other potential parameters which could influence drawdown capacity could be:

- current condition of the dam (10-yearly safety reviews under Section 10 of the Reservoirs Act may recommend rehabilitation or enlargement of the drawdown capability as one of the mitigation measures but it is more likely that they would recommend structural works to improve the condition)
- an estimated overall likelihood of internal erosion failure
- level of risk

The process to define the important deterioration factors at a dam are shown in Table 5.14, noting that the most important factors are likely to vary between individual dams. However, as the resources for a site-specific assessment are probably disproportionate for all except the very highest consequence dams, this section considers factors covering the majority of UK dams.

Table 5.14 Process to define deterioration factors governing drawdown capacity

Criteria		Sources of evidence that may support assessment for: an individual dam	Sources of evidence that may support assessment for: UK embankment dams generally
1	The critical and significant failure modes at the subject dam	Failure modes analysis using RARS (Environment Agency 2013)	Checklists in next section; Incident database (CIRIA 2014)
2	The geometry/material properties determining the vulnerability to failure, and its rate at the subject dam. These may be parameters governing rate of failure and/or parameters defining the threshold at which some critical shear stress or other factor is exceeded	Site-specific ground investigation and laboratory testing may be warranted	ICOLD Bulletin 164 (ICOLD 2013); *Draft engineering guide to early detection of internal erosion* (Defra 2007)
3	The evidence that the selected properties govern the above	Implicit by use of published methods for engineering analysis/to quantify likelihood of failure, including case histories and UK engineering guides	
4	The viability/reliability of measuring these properties	Experience of dam engineers	

5.8.3 Sources of long lists of candidate deterioration parameters

There are a number of existing engineering guides which include useful checklists of features to be considered in both dam safety reviews (including estimating the probability of failure) and surveillance visits, with the principal checklists summarised below. It is important to differentiate between the intrinsic condition of an element of the dam (what it was built of, and its condition when new) and its current condition (the state it is in now), with only the former likely to affect the vulnerability and speed of failure.

In addition to the assessment of the form of construction and materials used, the geometry of each element of a dam should be considered, as for example high dams with a narrow crest are likely to fail quicker than low dams with wide crests.

Other sources of long lists of potential parameters which have been considered, and used where appropriate, include:

- indicators used to define the level of a structural problem (incident) at a dam (although these may be the magnitude of symptoms, rather than the parameters governing dam behaviour)
- factors considered by others in defining the target rate of drawdown (see Section 2)
- parameters identified as important from records of incidents at dams (Environment Agency 2011, CIRIA 2014)

Table 5.15 Checklists used in defining deterioration factors for drawdown capability

Element of dam	Reference/checklist	Comments
Embankments/foundations	BRE Report 303 *Investigating embankment dams* (Charles et.al. 1996)	Limited to investigation of defects, rather than prediction of rate of failure
	CIRIA Report 161 (CIRIA 1996b) *Small embankment reservoirs*. Checklist on p.444	Both references provide design parameters (rather than parameters for rate of deterioration) and indicators (symptoms)
	BRE Report BR 363. *Engineering guide to the safety of embankment dams*. 2nd edition. Surveillance checklist in Table 8 (BRE,1999)	
	Environment Agency (2011) *Modes of dam failure and monitoring and measuring techniques*	Good qualitative description of failure modes. Monitoring parameters such as seepage and settlement are symptoms rather than parameters governing rate of deterioration
	Environment Agency (2013) *Guide to risk assessment for reservoir safety management* (RARS)	Candidate deterioration factors are those included in intrinsic condition (Boxes 8.17, 8.18)
	ICOLD (2013) Bulletin 164. *Internal erosion of existing dams, levees and dikes, and their foundations*. Volume 1: Internal erosion processes and engineering assessment	Various checklists of deterioration factors for the various phases for each of four types of internal erosion
Pipework	CIRIA (1997) Report 170. *Valves and pipework – guide to condition assessment*	
	Water industry research by WRc	
Concrete dams	CIRIA Report 148 (1996a) *Engineering guide to the safety of concrete and masonry dam structures in the UK*	
Concrete structures	Various general references on condition assessment of concrete (e.g. The Concrete Society 2000)	
General	Environment Agency annual post-incident reports	Although some useful case histories, no quantitative measure of onset or rate of failure

5.8.4 Criteria to screen deterioration features/parameters

The criteria used to select the factors for assessing drawdown capacity vary depending on how the information is used, for example whether semi-quantitative to classify the dam more/less vulnerable than the 'median dam', or whether a numeric value is used as part of an equation to quantify the drawdown capability. The criteria adopted are shown in Table 5.13, and comprise:

- relevance to time to onset of breach (rate of failure) (defined as end of Phase 7 in Table 4.2)
- ease of measurement
- viability of quantifying link between parameter value and rate of erosion

5.8.5 Screening of deterioration parameters

In Table 5.16, the parameters have been scored as described in Section 5.2. A total of 17 candidate factors have been identified, with the overall score varying between 5 and 9. Factors with as score of 8 or more are:

- hydraulic gradient
- the two parameters for rate of erosion

These parameters were therefore initially proposed to form part of a qualitative system for assessing required drawdown rate. As the guidance was developed the approach and parameters were refined. However, the hydraulic gradient and erosion rate index are fundamental parameters used in the guide to evaluate a dam's vulnerability to rapid failure.

Table 5.16 Screening of factors relating to the majority of UK embankment dams

Factor/parameter		Possible system(s) to quantify		Comment	Scoring (0 – not relevant to 5 – highly relevant)			Viability of quantifying link
Type	**Description**	**Parameter**	**Criteria for fast failure**		**Relevance to 'time to breach'**	**Ease of assessment**	**Total**	
Dam geometry	Hydraulic gradient from water line to downstream toe (backward erosion)	Hydraulic gradient	ICOLD Bulletin 164 Tables 4.3, 4.4	Also a measure of shear stress on concentred leaks. Could define 'critical gradient' for onset of erosion (or erosion> xxkg/hour) See Section 4 in Defra (2003) Stage B feasibility	4	5	9	Yes
	Crest width as % of reservoir height	Crest width		Surrogate for above	2	5	7	Yes
	Type of surfacing (if any) on dam crest	Look-up list			3	4	7	Yes
	Downstream slope angle	Angle	RARS Box 8.10, or 8.17		2	4	6	Yes
	Internal zoning (and other screening tests in Section 9 of ICOLD)		ICOLD Bulletin 164 Table 9.3	Questionable whether can apply to UK dams	5	1	6	Questionable
	Levels of points of weakness in dam	Level as % reservoir height		See Note 1	3	2	5	Difficult
Material properties – embankment	Concentrated leaks – erodibility	Erosion rate index	RARS Box 8.2 ICOLD Bulletin 164 Tables 3.4, 9.8		5	3	8	Yes
	Concentrated leaks – critical shear stress	Critical shear stress	ICOLD Bulletin 164 Table 3.5		5	3	8	Yes
	Contact erosion	D_{10} of soil	ICOLD Bulletin 164 Figure 5.2		3	2	5	Yes
	Suffusion	Several	ICOLD Bulletin 164 Section 6		3	2	5	Yes

Factor/parameter		Possible system(s) to quantify		Comment	Scoring (0 – not relevant to 5 – highly relevant)			Viability of quantifying link
Type	**Description**	**Parameter**	**Criteria for fast failure**		**Relevance to 'time to breach'**	**Ease of assessment**	**Total**	
	Downstream shoulder acts as filter	Look-up list	ICOLD Bulletin 164 Section 7		5	1	6	Yes
		Look-up list	RARS Box 8.17		5	1	6	Yes
Material properties – foundation	As embankment							
Material properties – culverts and pipes	Concrete vs brickwork	Look-up list			2	3	5	Probably
	For pipe laid directly in fill, the type of pipe material and joints				3	2	5	Probably
	Other screening issues as Box 8.18 of RARS	Look-up list						
	Annual probability of pipe fracturing			Difficult to reliability quantify	4	1	5	Yes
						Max	9	
	Number of candidate factors	17				Min	5	

Note:

1. Points of weakness could include levels of crest raising, depth of desiccation and change of geology up abutment(s), as well as construction singularities such as old crossing points over core, levels at which clay placing stopped one year and was resumed the next year. Records of incidents suggest these are often points at which leaks commence, and may be important in determining the level at which leaks stop as the reservoir water level is dropped.

5.8.6 Quantification of link between parameter and drawdown capacity

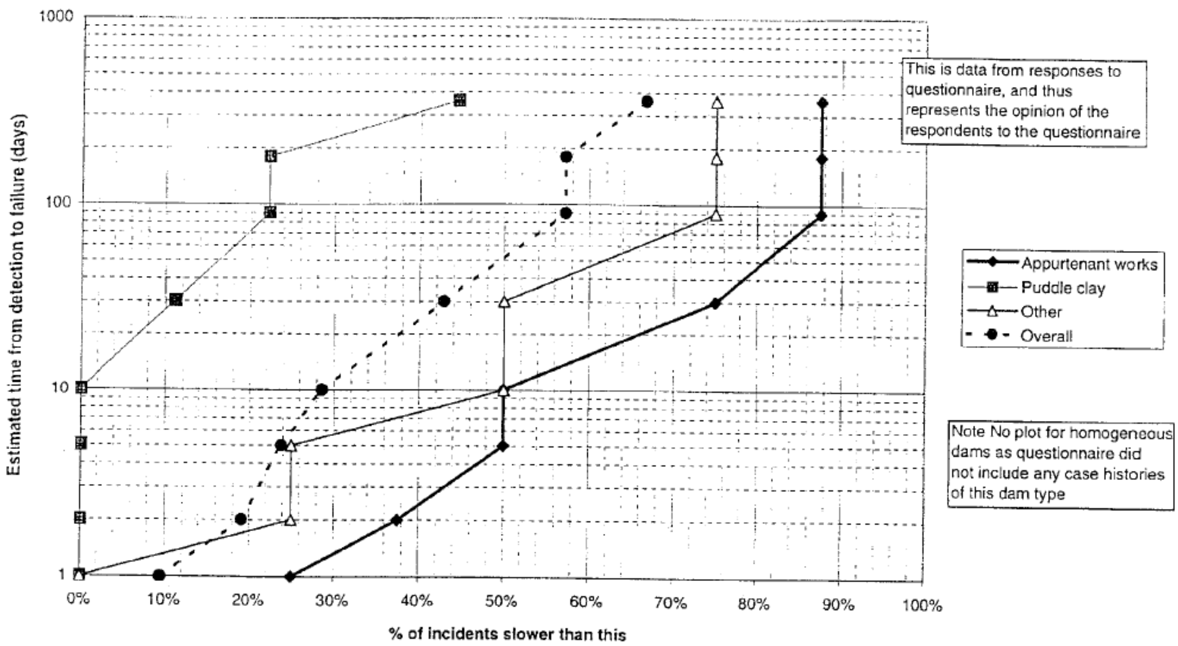
Tools that could be used to quantify a link between deterioration parameters and drawdown capacity include

- expert opinion
- analysis of process
- system model

Examples of these tools are described below, with their strengths and weaknesses.

Expert opinion includes the questionnaires and expert elicitation on early detection of internal erosion carried out by KBR in 2002–2007 under a Defra research contract, reported in the KBR feasibility report (Defra 2003) and Brown and Gosden (2004). An example of combined output from this research programme is shown on Figure 5.1.

Figure 5.1 Time to failure



The key advantages of this data is that it provides a direct link between time to failure and dam type. A disadvantage is that it is based on expert opinion, and thus does not provide a direct link to measurable soil parameters or dam geometry. The significant range of uncertainty should also be noted, reflecting the variability of both UK dams and opinion between individual experts.

Process analysis requires both a theoretical equation, and the data for the values of geometry and/or soil parameters used in the equation. Process equations are available in ICOLD Bulletin 164 on internal erosion (2013), and the Internal Erosion Toolbox (USBR 2008), with some of the tables in the latter reproduced in Table 5.17. The significant range on rate of erosion is noted, consistent with the range of time to failure in Figure 5.1.

Table 5.17 Extracts from Internal Erosion Toolbox (USBR, 2008)

Table 12.2 - Rate of Erosion of the core or soil in the foundation

Soil Classification	Best Estimate Erosion Rate Index (I_{HET})	Time for erosion in the core of the embankment or in the foundation	
		Gradient along pipe 0.2	Gradient along pipe 0.5
SM with <30% fines	<2	Very Rapid	Very Rapid
SM with > 30% fines	2 to 3	Very Rapid	Very Rapid
SC with < 30% fines	2 to 3	Very Rapid	Very Rapid
SC with >40% fines	3	Rapid	Very Rapid
ML	2 to 3	Very Rapid to Rapid	Very Rapid
CL-ML	3	Rapid	Very Rapid
CL	3 to 4	Rapid	Very Rapid to Rapid
CL-CH	4	Rapid	Rapid
MH	3 to 4	Rapid	Very Rapid to Rapid
CH with Liquid Limit <65%	4	Rapid to Medium	Rapid
CH with Liquid Limit > 65%	5	Medium to Slow	Medium

Table 12.3 – Influence of the material in the downstream zone of the embankment on the likely time for development of a breach.

Material Description	Likely Breach Time
Coarse grained rockfill	Slow – medium
Soil of high plasticity (plasticity index > 50%) and high clay size content including clayey gravels	Medium – rapid
Soil of low plasticity (plasticity index < 35%) and low clay size content, all poorly compacted soils, silty sandy gravels	Rapid – very rapid
Sand, silty sand, silt	Very rapid

Table 12.4 – Qualitative terms for times of development of internal erosion, piping and breach (Fell et al 2001, 2003).

Qualitative Term	Equivalent Time
Slow (S)	Weeks or months, even years
Medium (M)	Days or weeks
Rapid (R)	Hours (> 12 hours) or days
Very Rapid (VR)	< 3 hours

System models are more complex, comprising process equations combined to describe overall system behaviour. Dam break models are one such system model. These are still to some extent in the developmental stage, with stages of development in UK including BREACH, AREBA and EMBREA, each being developed to address weaknesses in the previous generation. These are often best used as sensitivity studies, to examine the sensitivity of output to defined ranges of input parameters, with one such sensitivity published in Section 2 of Defra FD 2658 (Defra/Environment Agency 2014).

Further details of how the link was quantified between deterioration parameters, the time it would take for failure to occur and the implications for drawdown capacity are explained in Section 8.

5.9 Other threats including actions of humans

Human actions (or inactions) that could lead to failure of a dam include those shown in Table 5.18. It is considered these should all be controlled by appropriate security and maintenance regimes, and that it would be inappropriate to use these actions to determine the size of drawdown facilities. With reference to Table 4.2 defining the framework for failure modes, this can be viewed as controlling the initiation phases of potential failure, rather than the progression phase.

Table 5.18 Human actions that could lead to dam failure

Type of action by humans	Preferred control	Link threat to size of drawdown capacity
Terrorist actions	Security/access to dam	No, as no direct link
Vandalism (e.g. shopping trolley/other debris blocking spillways or outlet)	As above, plus maintenance to mitigate/make good	More economical to carry out maintenance
Cut down trees on embankment, resulting in increased pore pressures and slope instability	If an embankment slope is potentially unstable, it is preferable/more economical to carry out structural works to stabilise the slope than to provide a larger outlet	
Excavation for services in toe of dam	Control by ensuring any works near/on dam are approved by a Panel Engineer before commencement	

Other potential threats should be considered on a dam-specific basis. A checklist of potential factors to consider is given in Table 8.9 of the RARS guide (Environment Agency 2013).

One such example is aircraft impact, which although being highly unlikely at most dams, may be a significant consideration at dams adjacent to airports. The risk of dam failure due to direct aircraft impact is best managed through increased freeboard and crest width (continuation phase), rather than provision of larger drawdown capacity (progression phase), as the time available for any drawdown capacity is likely to be insignificant compared to the time to failure.

5.10 Other considerations

Other considerations include the frequency of surveillance visits to the dam, the time it would take to activate drawdown ad the concurrent inflows into the reservoir at the time of drawdown. These factors are discussed in Volume 1 of the guide.

As a general rule, the guidance is based on the intrinsic condition of the dam (what it was built of, and its condition when new) without making allowance for its current condition (the state it is in now). This is because the intrinsic condition is likely to govern the vulnerability and speed of internal erosion more than the current condition. Also, it is assumed that if a dam is in poor condition this should be remedied directly and increasing the drawdown capacity should not be regarded as an alternative solution.

5.11 Summary of key factors

The previous subsections evaluated the list of threats identified in Table 7.2 of the RARS guide (Environment Agency 2013) and concluded that there are four main threats where emergency drawdown is an effective means of mitigation. These threats are listed below and need to be considered when evaluating drawdown capacity as discussed in Section 3.5 of the main guide:

- floods
- wind (waves)
- earthquake
- deterioration (internal erosion)

Of these four threats, the most critical is considered to be internal erosion and this is highlighted by the high number of reported incidents compared to the other threats (see Table 5.19). The threat of upstream dam failure was not considered a reasonable criterion for establishment of the drawdown capacity.

Table 5.19 Principal causes of UK dam incidents (taken from Figure 2.1 of CIRIA 2014)

Failure type	Number of failures (100 total)
External erosion – flood flow	21
Internal erosion/leakage on first filling	16
Internal erosion in service; ancillary works, cut-offs	16
Internal erosion/leakage in service	11
Slope instability during construction	9
Slope instability in service	8
Concrete/masonry dams	6
Basin leakage and instability	5
Pipe or valve failure	4
Wave damage to upstream protection	4

A total of 34 parameters were identified during the scoping and viability stages of the project which could potentially be used to assess the various threats, although some of these were duplicated. The screening process identified which of these parameters were most relevant, easiest to assess and could viably be related to time of failure and hence drawdown capacity. The shortlisted parameters which were derived through this screening process are summarised in Table 5.20. The fourth column of the table describes how the factors have ultimately been incorporated into the guide.

During the development of the guide, efforts were made to try to develop a quantitative system/tool kit to derive an ‘optimum’ drawdown capacity based on the parameters identified in Table 5.20. For each parameter quantified links were researched to determine its effect on the time to failure for each of the key threats. Draft guidance was developed on this basis but, following initial trials and industry consultation, the approach was rejected as it was considered too prescriptive and did not offer sufficient flexibility for judgement.

Table 5.20 Summary of shortlisted parameters

	Shortlisted parameter	Related threats	How it has been incorporated in Volume 1 of the guide
Parameters associated with threats in Section 5			
1	Crest width	Floods, wind, earthquake	This is considered to have been captured by the hydraulic gradient (see item 3 below).
2	Freeboard		Consideration was given to using freeboard as a parameter for assessing drawdown capacity but was rejected as it was difficult to make a quantitative link. It is still a factor that should be considered as part of the critical failure modes assessment in Volume 1, Section 3.5.
3	Hydraulic gradient	Deterioration (internal erosion)	This is a key factor in assessing the overall vulnerability of a dam to rapid failure in Volume 1, Section 6.4.2. Details of how the link was derived between hydraulic gradient and drawdown capacity are discussed in Volume 2, Section 8.7.
4	Surface erodibility of embankment fill		Erosion rate index (see item 5 below) is considered a reasonable surrogate parameter to also cover surface erodibility for the purpose of assessing drawdown capacity.
5	Erosion rate index of embankment fill (rate of erosion)		This is a key factor in assessing the overall vulnerability of a dam to rapid failure in Volume 1, Section 6.4.2. Details of how the link was derived between hydraulic gradient and drawdown capacity are discussed in Volume 2, Section 8.7.
6	Critical shear stress (onset of erosion) of embankment fill		This consideration is introduced in Section 6.4.2 and detailed in Appendix D.2 of Volume 1.
Other considerations			
7	Concurrent inflows		Volume 1, Section 5.2.
8	Frequency of surveillance		Volume 1, Section 6.5.1.
9	Activation time		Volume 1, Section 6.5.2.

Figure D.1 in Volume 1 of the guide provides a relationship between the theoretical drawdown rate required to avert failure and the two key factors affecting internal erosion; namely hydraulic gradient and erosion rate index. The derivation of this relationship is outlined in Appendix D of Volume 1 and is discussed further in Section 8.7.

6 Factors governing drawdown capacity for concrete dams

6.1 Introduction

This section is an assessment of the factors relevant to determining the preferred drawdown capacity for concrete dams. It complements Section 5 which covers the same issues for embankment dams. It does not repeat common material, but is limited to failure modes and parameters which are different from embankment dams. The term concrete dam is deemed to include masonry, as well as different types such as gravity buttress and arch dams.

In relation to the interface between concrete and embankment dams, these would normally be considered under embankment dams, being determined by the same factors governing behaviour between embankments and appurtenant structures such as spillways and outlet works.

6.2 Overview of the differences in failure modes between concrete and embankment dams

Screening of the various factors that could be used to size drawdown capacity in concrete dams is carried out in Table 6.1, with only ice and deterioration being carried forward to consideration of the parameters relevant to the sizing of drawdown capacity in concrete dams.

Table 6.1 Screening of failure modes to identify those which may be used to size drawdown capacity in concrete dams

Threat	Differences of concrete compared to embankment dams
Flood	Failure during a flood would be brittle, such that drawdown capacity is not relevant. If there was scour damage at the downstream toe, drawdown could be used to prevent further overtopping, with a parameter related to wet day flows as for embankment dams.
Wind	Unlikely to lead to failure (release of reservoir), so not warranted as a factor in sizing drawdown capacity.
Upstream dam	As embankment dams.
Ice	Effects of ice greater than embankment dams due to the vertical/near-vertical upstream face and the rigidity of the structure increases the effects of horizontal loading.
Earthquake	As for flood threat, failure during an earthquake would be rapid, such that the sizing of drawdown capacity would be governed by winter flows in the period until any repairs could be carried out.
Deterioration	Body of dam – loss of strength/watertightness (e.g. alkali–silica reaction, leaching of lime mortar). Drawdown has the potential to prevent subsequent failure and provide time to carry out repairs to strengthen the dam.
Other including actions of humans	As embankment dams.

6.3 Ice

Build-up of ice on a reservoir can lead to

- blockage of a spillway
- freezing of valves
- horizontal load onto rigid structures, the load depending on the thickness of ice (which depends on the number of degree days of freezing) and the rate of temperature rise (ICOLD 1996, USACE 2002)

Drawdown by more than the thickness of the ice sheet (the likely maximum thickness in the UK is 400mm) would have the effect of cracking the ice sheet and hence reducing thermally induced horizontal loads. In the case of loads generated by thermal expansion of the ice, overload resulting in cracking and displacement of part of the structure would partially relieve the ice load. Progression to failure may therefore require several applications of ice load. Drawdown could be used to reduce water levels and hence thermal ice loading effects, and/or reduce the load on the dam if displacement has occurred.

According to ICOLD (1996, p.59) even if the ice cover is cracked a rise in water level can impart a horizontal thrust on the dam from the interaction between floating blocks and blocks frozen to the face of the dam. It is suspected that this would only be significant for thicker ice sheets than are likely to occur in UK conditions, but drawdown would stop this effect.

Table 6.2 describes a candidate parameter which was initially identified for use in a quantitative method to determine drawdown capacity. Based on industry consultation such a quantitative approach was eventually rejected.

Table 6.2 Candidate parameters

	Possible system to quantify	Comment
Number of degree days of freezing per year	Ice load from ICOLD (1996)	Further research required on ice formation in UK to quantify impact

6.4 Deterioration

The process to identify deterioration parameters for concrete dams is similar to embankment dams as described in Section 5.8 and is not repeated here. The key published guide is CIRIA Report 148 (CIRIA 1996a), and the research on concrete dams by the Dam Safety Interest Group (DSIG, a subsidiary of CEATI International) (DSIG 2015).

Modes of deterioration and thus failure of concrete dams are given in Table 7.3 of CIRIA Report 148, with key modes including:

- alkali–silica reaction (ASR)
- blockage of relief wells, and build-up of uplift on failure planes
- leaching of grout curtains
- wash-out of joint infill in foundations

Most of these are slow processes which can be detected and managed through regular surveillance and maintenance, with structural intervention as necessary.

Table 6.3 shows how candidate parameters which could be used in a quantitative method for sizing drawdown capacity were screened. Based on industry consultation such a quantitative approach was eventually rejected but the principles were used in developing the guidance in Section 7.2 of Volume 1.

6.5 Summary of key factors

During the development of the guide, efforts were made to try and develop a quantitative system/tool kit to derive an 'optimum' drawdown capacity for concrete dams, based on the parameters identified in this section. This was found to be unviable and following industry consultation, it was agreed that Section 7.2 of Volume 1 should instead just describe the main considerations and factors that should be taken into account.

Table 6.3 Screening of factors which could be used to size drawdown capacity to prevent deterioration failure of concrete dams in the UK

Factor/parameter			Possible system(s) to quantify		Comment	Scoring (0 – not relevant to 5 – highly relevant)			
Type	Description	Failure mode	Parameter	Criteria for fast failure		Relevance to 'time to breach'	Ease of assessment	Reliability of quantifying link	Total
Dam geometry	Thickness/height ratio of dam	Stability failure	Gradient from water to downstream toe		Surrogate for degree of robustness of sliding/overturning stability	3	5	1	9
Material properties:									
Body of dam	Material properties, joints/cracks	Loss of strength/ watertightness (e.g. ASR, leaching of lime mortar)	Strength or condition of concrete/ masonry and joints			4	2	2	8
Foundation	Internal erosion/ redistribution of fines followed by build-up of uplift	Wash-out of joint infill/build-up of uplift	Hydraulic gradient along dam/foundation contact		Measure of gradient in rock joints	5	3	3	12
Structures	Outlet pipe fracture within dam, uplift pressures develop	Uplift	Vulnerability to lining failure			4	4	4	12
								Max.	12
		Number of candidate factors						Min.	8

7 Factors governing drawdown capacity for service reservoirs

7.1 Introduction

This section assesses the principal differences in failure modes between embankment dams and service reservoirs and any differences which affect required drawdown capacity.

Service reservoirs differ from raw water reservoirs in that complete emptying of the reservoirs is regularly undertaken for cleaning purposes. In most cases to avoid discharge of chlorinated water into watercourses the drawdown is undertaken into supply, or discharge via wash-outs on the supply main rather than wholly through a separate drawdown pipe discharging directly to a watercourse. This capacity will be termed the operational drawdown capacity.

7.2 Failure modes

Inflows: Service reservoirs are not subject to natural inflows but receive normally pumped inflows prior to entering the treated water distribution network. If a service reservoir is overfilled and inflows continue it is possible that the roof could be pressurised from within, fail and cause failure of wall sections. This mode of failure is usually prevented by overflow provision and alarms/lock-off systems to shut off the inflow. Drawdown capacity can play no useful part in preventing this failure mode as, if one is aware of the problem and able to act, the correct response is to shut off the inflow thus dealing with the problem at source.

Seismic: Similar considerations apply to the effect of seismic events as at embankment dams but the parameters which would be used to assess the required drawdown capacity would be different. The different format of damage (i.e. failure of supporting embankment, structural failure or failure resulting from leakage) are no different from those considered in the following three sections and separate consideration is not required.

Excavation into perimeter embankment: This would potentially result in loss of the supporting fill. If this were sufficient to itself cause structural failure there would be no time to react and drawdown would not be effective. If it required further degradation to the supporting fill to cause failure, drawdown would reduce the load. This would be likely to take several days and be a result of weather and time. The operational drawdown capacity would be sufficient.

Deterioration of material forming body of structure leading to collapse: Structural failure of a service reservoir once there are clear signs of distress would be sudden and there is unlikely to be any warning time to effect an emergency drawdown. This failure mode should be managed by earlier intervention when a precautionary drawdown could be undertaken. The required rate for this would be the same as routine drawdown for operational purposes (e.g. periodic cleaning).

Deterioration of material forming body of structure leading to leakage: This failure process would follow the same path as internal erosion in an embankment dam, although depending on the source of the leakage the hydraulic gradient may be higher. Apart from this possibility needing to be evaluated, the approach adopted would be similar to that applied to embankment dams.

Deterioration of foundation (not due to leakage): The likely failure mode from this threat is differential settlement of the structure leading to leakage from the reservoir. This could potentially develop to failure through internal erosion and has already been considered above.

Deterioration of pipework: The failure path from this would be through leakage causing internal erosion of the foundation or supporting fill. This has already been considered above.

Operational drawdown: When service reservoirs are taken out of use for cleaning, the duration of this will normally be minimised. Typically the time to drawdown an individual cell of a modern service reservoir is less than a day but older reservoirs may be up to a week. Where failure can be averted by emergency drawdown, complete emptying of a reservoir in this time will frequently be sufficient.

7.3 Conclusions

Service reservoirs normally have a means for relatively rapid emptying of the contents for operational purposes. This will commonly be less than one day for any individual reservoir cell.

Internal erosion following a leak is the principal failure scenario where emergency drawdown could be effective in averting failure. The approach adopted for evaluation of this at embankment dams should also be appropriate for service reservoirs, as long as the potentially higher hydraulic gradients (depending on the location of the leakage) are considered.

The methodology developed in Section 7.3 of the main guide combines the approaches for embankment dams and concrete dams.

8 Review of the time to failure for UK dams

8.1 Introduction

8.1.1 Background

A key parameter in determining an appropriate drawdown rate is the time it would take for the dam to fail, from the point when a defect becomes detectable, to the point when catastrophic failure and uncontrolled release of water is unavoidable.

The following section is closely based on a paper which was prepared at a stage during the development of the guide. At the time the paper was produced it was intended to develop a numerical system which would allow users of the guide to derive drawdown rates commensurate with the predicted time it would take a dam to fail. The numerical system was not adopted in the final version of Volume 1 of the guide (except to produce Figure D.1) but the principles discussed are still pertinent to assessing drawdown capacity.

A review was carried out of the various methods available to predict the time it would take a dam to fail in order to identify a suitable method to incorporate within the guidance. This section summarises this review and applies the selected method to a range of six typical UK dams. The actual times observed in real incidents are also reviewed and the robustness of the selected approach is assessed.

Since the majority of reservoirs in the UK are impounded by embankment dams this was the prime focus of the review. The most common cause of reservoir failure in the UK is internal erosion (43% of the incidents in Table 5.19) and the time to failure considered in this section is based on this failure mechanism. While another major failure mechanism is external erosion caused by overtopping flows, this was not taken into account for the purpose of assessing time to failure in the review. This is because overtopping failure is generally the result of flood flows and it would be unrealistic for drawdown facilities to be sized to pass such high flows. Passing flood flows is the function of a spillway and it is not the purpose of a low-level outlet to duplicate this function. Similarly, other threats (e.g. waves, ice, earthquakes) are either of rare occurrence, or are better managed by other means.

8.1.2 Approach

There is a range of published information which could contribute to the means of predicting an approximate time to failure for a given reservoir. This information is discussed in Section 8.3 and includes, empirical and theoretical models, expert opinion, software models and physical tests. Four key criteria were applied to assess the suitability of these tools for use in the guide as follows:

- Simplicity – To ensure that the guidance is widely used, any method adopted within it to predict the time to failure needs to be relatively simple and easy to follow. It needs to be based on commonly known parameters.
- Repeatability – The method should be based primarily on quantifiable parameters with limited reliance on user judgement.

- Validity – The method should be linked to the physical processes of internal erosion and the key influencing parameters. In particular, in order for the method to be applied as part of the guidance, it needs to be directly related to drawdown rate, or in other words hydraulic gradient.
- Accuracy – This will be assessed by comparing the method with reports of and actual incidents at dams.

The different approaches currently available use slightly different terminology when discussing the time to failure and soil erodibility. This is discussed in Section 8.2 in order to establish clear terminology for use in this report.

8.2 Concepts and definitions

8.2.1 Mechanisms of internal erosion

ICOLD Bulletin 164 (ICOLD 2013) defines four mechanisms of internal erosion as follows:

- **Concentrated leaks:** Such as leakage through a crack caused by differential settlement.
- **Backward erosion:** There are two types of backward erosion:
  - backward erosion piping, which typically occurs in foundations, whereby a ‘pipe’ erodes from the downstream side of the dam and works back
  - global backward erosion which leads to development of a near-vertical pipe in the core of an embankment.
- **Contact erosion:** This occurs where a coarse soil is in contact with a fine soil and flow parallel to the interface erodes the fine soil.
- **Suffusion:** This occurs when water flows through internally unstable widely graded or gap graded non-plastic soils. The small particles of soil are transported through the pores of the coarser particles.

This section mainly considers concentrated leaks and backward erosion piping through the embankment or at the interface with the foundation.

8.2.2 Stages of failure

ICOLD Bulletin 164 defines four phases in the process of internal erosion as described below and this model will be followed in this report:

- **Initiation**: Concentrated leak forms and erosion initiates along the walls of the crack, or in the case of backward erosion, at the exit point of the leakage.
- **Continuation**: The erosion continues.
- **Progression**: The concentrated leak enlarges, or in the case of backward erosion it progresses to form a pipe.
- **Breach**: Breach mechanism forms.

The term ‘time to failure’ is used in a different sense in different contexts. Although this section focuses on the time to failure caused by internal erosion, it is useful to also consider the definition in terms of overtopping as well, and some of the stages between initiation and failure for both failure mechanisms are summarised in Table 8.1. It is recognised that this is a simplification for the purpose of quantifying the time to failure.

Table 8.1 Stages between initiation and breach

	Phase in development		**Terminology used in**	
	Internal erosion (for concentrated erosion – see Note 1)	**Overtopping**	**Internal erosion probability models**	**Hydraulic modelling of overtopping** (Note 2)
1	Initiate	Water starts to flow over crest	Initiation	I Initiation of overtopping
2	Stopped by filters etc.	Stopped if non-erodible face/crest cover layer	Continuation, if unfiltered	Continuation if velocity > allowable
3a	Detectable		Progression	II Headcut formation (Note 3)
3b	Roof forms	Overflow develops into a headcut (Note 3)		
3c	Hole enlarges (breach initiation)	Headcut migrates from downstream to upstream edge of crest		Transition to breach – steady (and relatively slow) erosion (Note 4)
3d	Roof collapses with sinkhole above water line			
4a	Hole breaks through to reservoir/roof collapses within reservoir	Crest starts to lower	Breach	Breach formation III Headcut migrates into reservoir and ends when downward erosion stopped
4b	Breach deepens			
4c	Breach widens			IV Peak discharge
4d	Breach fully developed. In large reservoirs water level only drops in this stage			

Notes

1. Type of progression will vary depending on the type of internal erosion – see the *International levee handbook* Section 3.5.2.2 (CIRIA 2013) for description of four types. This table has been prepared for concentrated erosion.
2. The model indicated above is taken from FEMA (2007), which cites Hanson et al. (2003) and Wahl (1998), and Section 2.3 of FLOODsite report T06-06-03 (Morris 2009).
3. Overtopping erosion occurs by headcutting in silts and clays, surface erosion in sands and gravels, and loss of interlocking in rockfills (Table 2.3 of FLOODsite T06-06-03, Morris 2009).
4. FEMA 602 Section 2.4.2 notes that in overtopping failure the initiation time can be quite lengthy and often greater than the breach formation time. Morris notes the time period in the early stages of progression can be hours, days or months.
5. The highlighted stages (i.e. from detection (3a) to breakthrough (4a)) represent the 'time to failure' period as defined and used in this review.

In relation to overtopping failure FEMA (2007) differentiates initiation (Stages I and II) from breach (Stages III and IV). The processes that are considered to be important in breach prediction due to overtopping are summarised in Figure 8.1, and comprise both hydraulic issues of flow through/over the dam, and geotechnical parameters affecting the rate of erosion.

Figure 8.1 Processes involved in modelling breach due to overtopping (Morris 2009)

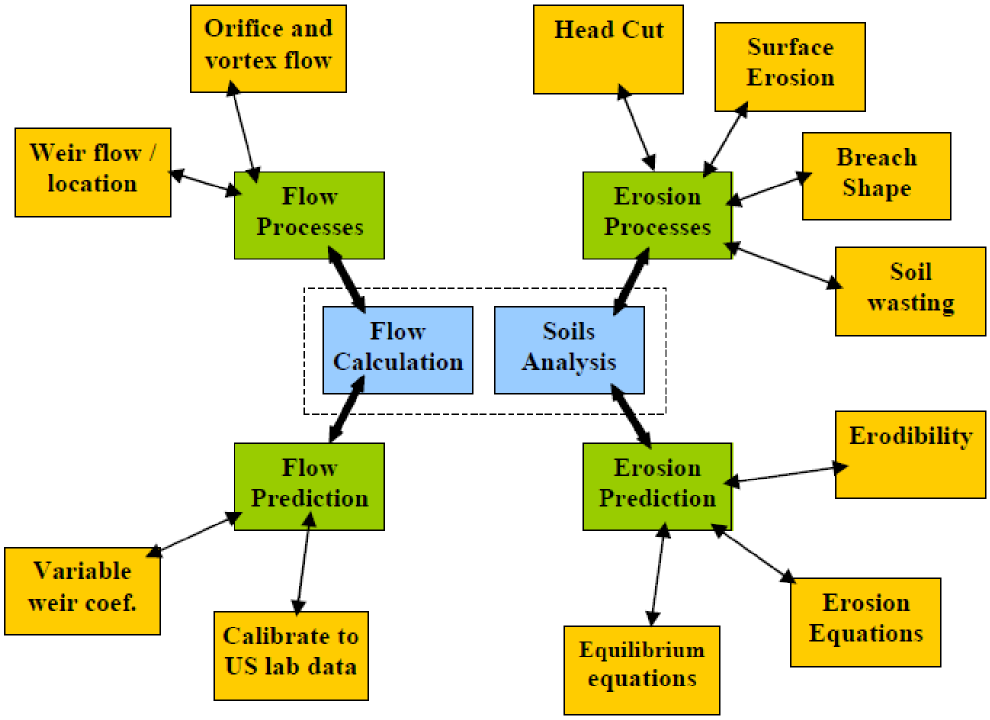


Figure 2-7 Schematic plan showing research and development topics for the HR BREACH model

ICOLD Bulletin 164 (ICOLD 2013) indicates that there are a number of similar issues which apply to models of a breach due to internal erosion, with key aspects comprising:

- the physical mechanisms change through the breach process. Internal erosion commences with erosion due to surface shear (initiation, and then the rate of erosion), but as the erosion hole enlarges the mechanism includes blocks of material dropping from the roof (or sides) of the erosion hole
- the rate of erosion will accelerate as time progresses, due to the enlarging hole
- peak breach flow is only likely to fully develop in Stages 4c or 4d.

Further discussion on breach models is provided in the *International levee handbook* (CIRIA 2013) and this is referenced in Table 8.2.

Table 8.2 Guidance in the *International levee handbook* (CIRIA 2013) on models of breach due to internal erosion

Section	Title	Comment
3.5.2.2	Main process of deterioration, damage and breach	Four types of internal erosion
8.3	Internal hydraulic processes	Limited to elastic behaviour
7.8.3.6 (p.635)	Erodibility	Limited to list of tests. No typical values included

8.2.3 Definition of time to failure

The purpose of drawdown facilities is to lower the reservoir level sufficiently such that the shear stress in any erosion hole no longer exceeds the critical shear stress in the hole, or in very erodible soils where once erosion has started it will continue under the smallest head, to lower the reservoir to below the level of the erosion hole. Thus the point at which a low-level outlet would no longer be effective at preventing failure will vary with soil type, but in general will probably correspond to the onset of breach.

Thus the time that is relevant to the capacity of low-level outlets may therefore be better termed the 'time to breakthrough', corresponding to the time from detection (3a) to breakthrough (4a).

The 'time to breakthrough' is the period that will be considered as the term 'time to failure' throughout the remainder of this review and is highlighted in Table 8.1. It is also illustrated in Figure 8.8 which plots the inferred rates of flow during an actual failure incident. By the time breakthrough has occurred, catastrophic breach is likely to occur within a very short period (i.e. see point C on Figure 8.8) and no amount of intervention could save the situation.

8.2.4 Detectable seepage

Methods for assessing the time to failure by internal erosion typically assume an initial pipe diameter; this accounts for both modelling and some other methods such as Figure 8.1 in ICOLD (2013). The latter method shows the time for a pipe to enlarge from 25mm to 1m diameter, although it is stated that for a pipe to further erode to 2m would take approximately 20% longer (ICOLD 2013). This indicates that the rate of erosion is exponential and means that identifying the failure at an early stage is important for averting failure. Computational models assessed in this review assume a similar range of initial pipe diameter, ranging from 10 to 50mm. If the rate of erosion is exponential the assumed initial pipe diameter could have a large effect on the overall time to failure.

Expert elicitation has previously considered this issue and the published results (Brown and Gosden 2004) suggest that the lowest rate of seepage which would be detectable is likely to be approximately 2l/minute. To equate this seepage rate to an initial pipe diameter, simple calculations have been carried out for different sized pipes. The pipe length was based on the mean dam height from the BRE Dam Register (1994) of 10.3m, assuming the embankment slopes are 1:3, the crest width is 3m and that the 'pipe' takes the shortest path at the base of the dam. The reservoir was assumed to be full and to allow for soil friction along the seepage path, a roughness coefficient of 3mm was assumed.

It was found that the minimum detectable seepage flow of 2l/minute suggested by the expert elicitation would equate to a pipe diameter of around 10mm under these conditions. Similarly, a 25mm diameter pipe would discharge approximately 22l/minute which expert opinion suggests would have been previously detectable for some time before it reached this stage.

Clearly the relationship depends on many factors such as the head across the pipe but it appears that, for an average dam, the time to failure predicted by many of the available methods may underestimate the time to failure based on the definition in Section 8.2.3 (i.e. indicate a more rapid failure as they are based on a period starting sometime after the seepage first becomes detectable).

For the purpose of deriving the relationship in Figure D.1 of Volume 1 of the guide, the 'time to failure' was assumed to commence from an initial hole size of 5mm which represents the point at which a concentrated leak may first be detectable.

8.2.5 Amount of damage that can be sustained without failure

The other end of the definition of time to failure is defined as 'breakthrough' which is based on the maximum amount of damage a dam could sustain before lowering the reservoir will no longer be effective in preventing failure. In terms of the quantity of leakage through the dam Figure 2 of Fell et al. (2001) is reproduced as Figure 8.2 and defines the extent of internal erosion that may occur, depending on the grading of any filter zone, with only continuing erosion leading to failure. They state 'the criteria for the excessive erosion boundary are selected from the case studies, and dams which experience erosion to this limit may have large piping discharges – up to say 1 to 2m^3/s. Whether a dam can withstand such flows without breaching depends on a number of factors including the discharge capacity of the downstream zone, and whether unravelling or slope instability may occur'. Thus it is considered that the boundary of performance when lowering the reservoir level could be beneficial, includes situations with leakages up to around 1 to 2m^3/s.

Figure 8.2 Filter erosion boundaries (reproduced from Figure 2 of Fell et al. 2001)

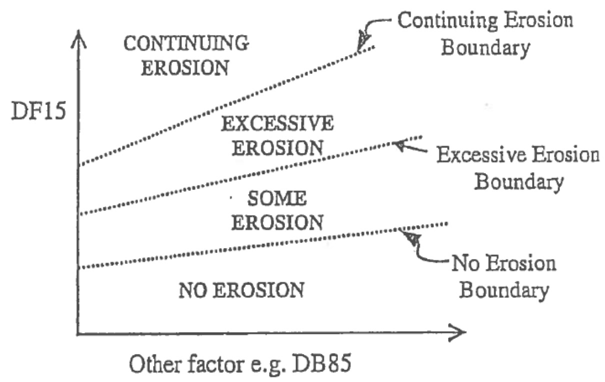


Figure 2 Filter erosion boundaries (Foster 1999)

A similar value is available from experience with internal erosion in cores built of broadly graded glacial soils in Sweden (Bartsch and Nilsson 2007), where sinkholes and leakages have included observed flows of 0.2m^3/s at Suorva dam in Sweden. Bartsch and Nilsson also note that 'at the Buliileo dam in Chile reports of leakage that under longer periods of time were of the order of 1m^3/s and experienced short team peaks of up to 8m^3/s. At this dam there was no filter between the sealing layer and the rockfill shoulder'.

As before it is useful to equate the typical 'breakthrough' flow of 1 to 2m^3/s to hole sizes in the dam and, to do this, similar assumptions have been made regarding the geometry of a typical UK dam as stated in Section 8.2.4 (i.e. embankment slopes of 1V:3H, a crest width of 3m and a roughness coefficient of 3mm). The 5th, 50th and 95th percentile dam heights from the BRE Register of British Dams (1994) were considered, assuming the dam was full in each case. Two piping scenarios have been considered, for piping two-thirds up the dam, and piping at the base of the dam. The results are shown in Table 8.3.

Table 8.3 Hole sizes corresponding to 1 to 2m^3/s discharge

Dam height (m) (percentile within BRE Dam Register, 1994)	**Hole diameter (m) equating to 1 to 2m^3/s discharge**	
	Hole at base of dam	**Hole two-thirds of the way up**
2 (5th percentile)	0.58–0.78	0.69–0.95
7 (50th percentile)	0.52–0.68	0.57–0.76
28 (95th percentile)	0.49–0.64	0.51–0.67

Table 8.3 indicates that, for the above definition of time to failure, the pipe diameter at breakthrough would be in the order of 0.5 to 1m for most dams. On average this is perhaps slightly less than the hole size of 1m given in ICOLD Bulletin 164 (see Section 8.3.4) but overall a hole size of 1m is probably reasonable as a round number, especially since hole size accelerates exponentially over time so the difference in terms of time between a 0.5m diameter hole and a 1m diameter hole is probably relatively small.

8.2.6 Relevance to overtopping or slope instability failure

In terms of slope instability or scour from overtopping flows, failure will occur when a slip or headcut extends across the dam to the waterline. At the Lower San Fernando dam in 1971 where the dam crest slumped 30 feet (about 9m) to just above current water level in a magnitude 6.7 earthquake the dam did not fail as a minimal freeboard remained. In terms of scour of parts of the downstream face, the scour (or loss of embankment section) would need to extend across the crest and down to the water line before failure. Some examples of loss of embankment section are shown in Table 8.4.

Table 8.4 Examples of embankment scour failure

Dam	Loss of embankment section
Boltby  Boltby 20th June 2005. Damage to toe of dam after storm on 19th.	Scour down mitre and at toe. Crest not compromised.
Ulley 	Scour of downstream face, extending half way up face. Possible tension cracks on downstream side of crest. No loss of crest.

In these situations of slope instability or scour from erosion of the downstream face a bottom outlet would assist in lowering the reservoir when there had been some loss of embankment section, with failure only inevitable (and the bottom outlet of no value) when the loss of section (scour and/or slope instability) has extended across the crest to the water line.

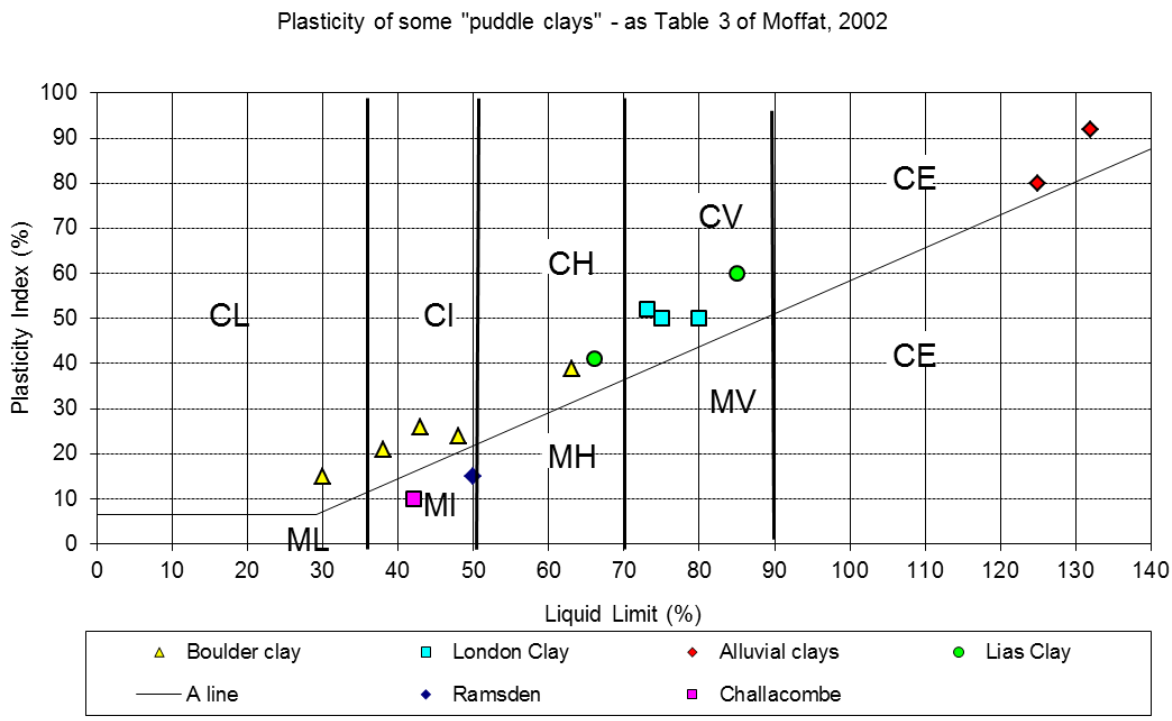
Although incidents such as those in Table 8.4 may not progress to failure, it would still be desirable to draw the reservoir down after such events to make repairs. The loss of embankment section will have increased the hydraulic gradient and thus increased the risk of internal erosion, therefore the concept of using time to failure by internal erosion as a means of determining an appropriate drawdown rate is still considered a reasonable approach for these types of scenarios.

8.2.7 Range of soils in UK dams

Moffat (2002) provides a useful review of the range of clays used in ‘puddle clay cores’, as summarised in Figure 8.3, where it can be seen that it varies from low plasticity clays and silts to extremely high plasticity clays. Laboratory erosion tests on soils have shown that there is a broad relationship between soil erodibility and plasticity with the rate of erosion varying by several orders of magnitude. This suggests that there will be a large range in the erodibility of clay in UK dams.

It should be noted that currently there is little evidence of dispersive clays in the UK, which would be prone to very rapid failure. Thus the very rapid time to failure reported where dispersive soils are present in tropical and arid climates is unlikely to be applicable to UK conditions.

Figure 8.3 The range of fine-grained soils used in 'puddle clay' dams (after Moffat 2002)



8.3 Review of relevant literature

8.3.1 Overview

This section summarises the current published literature related to rates of internal erosion and methods to calculate a time to failure. The methods are compared and assessed later in this section. There are four broad types of published methods for calculating time to failure as summarised in Table 8.5.

Table 8.5 Types of information available which could contribute to predicting time to failure

Type	Comments
1. Processes defined in guidance/papers	Much of the work in this field has been carried out by the soil mechanics department at the University of New South Wales in Australia, under the direction of Professor Robin Fell. Consequently, there are similarities between several of the published papers which effectively draw on the same research. The most up-to-date version of Fell's methods have been published in the Internal Erosion Toolbox (USBR 2008). A complementary method, which links the time for pipe enlargement with erosion rate index and hydraulic gradient is presented in Figure 8.1 of ICOLD Bulletin 164 (ICOLD 2013).
2. Published data from expert opinion	This includes two sources of work: • Expert elicitation carried out in 2004 (Brown and Gosden 2004) as part of a Defra research project using a panel of 11 dam experts whose responses were weighted based on how accurately they answered seed questions. • A questionnaire to a wider group of 40 reservoir panel engineers, carried out in 2003 as part of the same Defra research project as above.
3. Software models	These are discussed in Section 8.3.5. Much of the software is still relatively developmental and not yet commercially available.
4. Physical tests (laboratory and/or field)	The literature review has found very little published data on time to failure from actual physical testing of internal erosion, with the only reported tests as follows: a) Large-scale physical models of earth embankments tested to simulate internal erosion and breach widening (Hanson et al. 2010). b) Field trials carried out 2001–2004 as part of the Impact Project http://www.impact-project.net/. c) Field trials carried out as part of ongoing Dutch field trials http://www.ijkdijk.nl/en/.

A summary of the literature review is presented in Table 8.6. The key methods are described and discussed in more detail in the following subsections.

8.3.2 Common equations for soil erodibility

Soil erodibility is most commonly described in the literature by the excess stress equation (see Table 8.7, Note 1). This defines the rate of erosion based on an erodibility coefficient and the difference between the actual shear stress applied to a soil and the critical shear stress required to detach individual soil particles. The relationship is commonly expressed as the rate of erosion in either terms of volume per unit area per unit of time or mass per unit area per unit of time. This equation and other common equations relating to soil erodibility are given in Table 8.7. A key parameter in the excess stress equation is the erodibility coefficient, or 'detachment rate' k_d. This coefficient is also widely used in other sediment transport equations (e.g. for streambeds and streambanks).

Table 8.6 Summary of literature review on time to failure

No.	Reference	Provides information relevant to quantifying the following stage in the breach process: 1 Initiation	2 Continuation	Progression: 3a Detection	Progression: 3c Surface erosion	Progression: 3d Bulk removal	Overall time to breakthrough/ failure	Typical parameters for use in analysis	General comments	Conclusions (relevant to time to failure)
	Defined processes – papers based on work by Fell									
1	UNICIV Report R-399 (Fell et al. 2001)	Refers to Foster and Fell (1999, 2000)	Boundary of excessive/ continuing erosion may have large piping flow discharges (up to 1m³/s)	Section 4, Accidents normally detected in the progression, rather than initiation or continuation phases	Table 10 is stated to be a method but the logic is unclear. The method is based on combining the likelihood of 4 factors (Tables 5 to 7, 9) to give one of four categories				Table 3 provides evidence (i.e. details of case histories).	Table 10 provides a method of establishing index of time to failure (i.e. slow, medium, rapid or very rapid), these are linked but have a limited impact on output value
2	Internal Erosion Toolbox (USBR 2008)	Section 9 assesses the probability of various forms of internal erosion	Section 10 evaluates probability based on a range of factors such as filters	Section 12 includes the probability of detection this uses time to failure as one of the variables	Table 12.1 approximate time for progression and breach development			Soil classification, percentage fines, moisture condition, dam zones and material description	Focus is on probability of failure but does include a method for approximating time to failure	Table 12.1 can be used to approximate time to failure
3	Time for development of internal erosion and piping in embankment dams (Fell et al. 2003)	Different means of initiation listed		Guidance on detection	Time to failure table similar to Table 12.1 in the Internal Erosion Toolbox (USBR 2008)			Soil classification, percentage fines, moisture condition	This report sets up the basis of the approach used in the Internal Erosion Toolbox (USBR 2008)	More suitable to use the Internal Erosion Toolbox (USBR 2008)
4	Experimental investigation of the rate of piping erosion of soils in embankment dams, Wan et al. (2002)	Not related to time to failure							Initial shear stress τ_0 possibly being a another tool to calculate erosion rate index instead of τ_c	Not relevant to time to failure but does indicate methods to calculate the erosion rate index of a soil sample
	Defined process – ICOLD Bulletin 164									
5	Concentrated leak	Covered in Section 2.3	Sections 2.4 and 7 give guidance on evaluating effectiveness of filters. Nothing on rates	Covered in Section 2.6	3.3 suggests crack widths; 3.4 provides theoretical method to estimate surface erosion; Figure 8.1 shows time to erode to 1m diameter pipe, based on soil erosion index and gradient	Nothing on rate of upwards progression of roof (sinkhole formation)	Page 119 (11.2.2)	Covered in Section 9.7	Provides guidance on the physical process and vulnerability, but little on rate of erosion	Figure 8.1, could be reproduced and extended for a wider range of input factors (hole diameter and gradient). Nothing for other types of erosion
6	Backward erosion				4.4 to 4.6. Guidance limited mainly to initiation; critical gradient very sensitive to soil type					
7	Contact erosion				Nothing on rate					
8	Suffusion				Nothing on rate					
	Expert opinion – Defra Task B									
9	Expert elicitation (Brown and Gosden 2004 and Brown and Aspinall 2004)	Not covered	The 'calibrated decision maker' (see ninth column) considered that around 8% of puddle clay core dams and 4% of homogeneous have ongoing leakage, of which 14% have ongoing internal erosion rate of 25 grams/day	Q32–33 Minimum detectable flow 2 litre/minute on grass, 10 litre/minute on scrubby face	Q47–51 – Impact of hydraulic gradient, soil properties and degree of compaction on time to failure Q58–69 – Split of types of internal erosion Q70–71 – Progression of internal erosion		Q34–36 Leakage flow when failure inevitable 0.4m³/s. Q37–46. Median time from detection to failure 7 hours (range 1 to 135) on puddle clay, 3 hours (range 2 to 142) on homogeneous	Indirect link as some of the questions relate to the effect of different soil properties	60 questions on issues related to internal erosion. Opinion of 11 experts, each giving 5%, 50% and 95% confidence limits on response. Large range of opinions between individuals. Individual opinions weighted according to quality of response to 11 seed questions, the output being that of 'calibrated decision maker'	Probably best used as data to derive/ calibrate/compare any analytical method, rather than as a tool itself
10	Questionnaire to Panel Engineers (Defra 2003, Task B, Vol 2 Appendix C, Annex column)	Not covered		Q19–22d Q40–49	Q23–28		Q5–55 – actions taken to control	Indirect link as some of the questions relate to the effect of different soil properties	Industry questionnaire. 40 respondents to 115 questions Q56–57 provide information on available drawdown capacity	As elicitation (see row 9 above)
11	Calculation of rate of erosion in circular hole (Defra 2003)	Not covered			Sensitivity study using Wan et al. (2002) approach. Similar to Figure 8.1 in ICOLD Bulletin				Figures 4.10 and 4.11 of Defra (2003). Assumptions described in Section 4 of main report	Could be used as part of tool to adjust for gradient and erosion rate index
	Physical testing – Hanson et al.									
12	Hanson et al. (2010)	Not covered			Two full-scale embankment tests carried out but no method proposed to calculate time to failure, see conclusion			k_d	Summarises equations for soil erodibility and the relationship between different erodibility coefficients	Two tests carried out on different soils: 1. k_d = 100cm³/N/s failed in around 15 minutes 2. k_d = 0.1cm³/N/s test terminated after 72 hours before breakthrough occurred

Note: This table excludes software models which are covered separately in Section 8.3.5

Table 8.7 Common equations for soil erodibility

Equation no.	Details	Equation	Reference	Comments
1	Soil erodibility expressed as volume per unit of time	$\boldsymbol{E_r = k_d(\tau_e - \tau_c)^\alpha}$ E_r= the rate of erosion (m/sec) k_d= the detachment/erodibility coefficient (cm^3/N/s) τ_e= the effective stress (N/m^2) τ_c= the critical stress (N/m^2) α = exponent (sometimes assumed as 1)	Hanson et al. (2010)	These equations are known as the excess stress equations (Note 1)
2	Soil erodibility expressed as mass per unit of time	$\boldsymbol{E_t = C_e(\tau_e - \tau_c)^\alpha}$ E_t= the rate of erosion (kg/s/m^2) C_e= detachment/erodibility coefficient (s/m)	Hanson et al. (2010)	
3	Detachment/ erodibility coefficient	$\boldsymbol{C_e = k_d/\rho}$ k_d=the detachment/erodibility coefficient (cm^3/N/s) ρ=dry density	Hanson et al. (2010)	Note 2
4	Erosion rate index	$\boldsymbol{I_{HET} = -\log C_e}$	ICOLD (2013) and Hanson et al. (2010)	This is used in ICOLD (2013) Figure 8.1. It can be measured using the slot test or hole erosion, or jet erosion test
5	Detachment coefficient	$\boldsymbol{k_d = \frac{10\gamma_w}{\gamma_d} exp\left[-0.121(C\%)^{0.406}\left(\frac{\gamma_d}{\gamma_w}\right)^{3.10}\right]}$ k_d= the detachment/erodibility coefficient (cm^3/N/s) C% is clay percentage γ_d is the dry unit weight γ_w is the weight of water	ISIS help file (Innovyze 2013) (Note 3)	
6	Erodibility coefficient, Csecs	$\boldsymbol{Csecs = \frac{k_d}{3600}}$ Csecs is not given any units (See Note 4)	Info Works help file	Used in AREBA software

Notes:
1. The term ‘excess stress equation’ is used in various papers and text books such as Fell et al. (2003), Habersack et al. (2007), Hanson et al. (2010), Al-Madhhachi (2012).
2. Equation 3 is referenced like this in several sources but may be incorrect because when the units for k_d are divided by those for ρ they do not match the units of the erodibility coefficient. The units would be correct if k_d and ρ were multiplied instead of divided.
3. The full help file is not available but an extract from it was reproduced in the Overwater dam break study (classified report for United Utilities).
4. The Overwater dam break study also gave an alternative approach for calculating Csecs based on compactive effort but noted that the value obtained was far smaller than the preferred approach using Equation 6 above.

Table 8.7, Equations 3 and 4, show how k_d is related to erosion rate index which is another common parameter used to determine the time to failure (e.g. ICOLD 2013, Figure 8.1). Table 8.8 shows a comparison of the two parameters over a typical range of dry densities.

Table 8.8 Comparison of k_d, C_e and erosion rate index (Hanson et al. 2010)

k_d (cm^3/N-s)	[1] C_e (s/m)	[1] I = -log(C_e)	[2] Group No.	[2] Description
1000	0.5 – 0.7	0.2 – 0.3	1	Extremely Rapid
100	0.05 – 0.07	1.2 – 1.3	1	Extremely Rapid
10	0.005 – 0.007	2.2 – 2.3	2	Very Rapid
1	0.0005 - 0.0007	3.2 – 3.3	3	Moderately Rapid
0.1	0.00005 – 0.00007	4.2 – 4.3	4	Moderately Slow
0.01	0.000005 – 0.000007	5.2 – 5.3	5	Very Slow
0.001	0.0000005 – 0.0000007	6.2 – 6.3	6	Extremely Slow

[1] Based on range of ρ from 1500 to 2000 kg/m^3.
[2] Group No. and qualitative description grouping based on Wan and Fell (2004).

8.3.3 Internal Erosion Toolbox

The Internal Erosion Toolbox (USBR 2008) publishes the latest work developed by Fell et al. and allows the user to calculate the risk of a dam failing due to internal erosion. The risk is calculated by analysing a range of internal erosion failure mechanisms and their associated probabilities of failure.

One of the stages within the toolbox (Section 12) estimates the probability of an internal erosion mechanism being detected with subsequent intervention and repair. One of the variables used for this assessment is the time from initiation to dam breach, and the toolbox provides a series of tables to allow users to estimate this period based on basic geotechnical parameters, principally the soil plasticity and grading. The time to breach is defined as beginning when ‘a concentrated leak is first observed’, which agrees with the definition used in this paper.

Certain panel engineers within the UK have reported issues when applying the Internal Erosion Toolbox for UK dam risk assessments. They have found that some of the methods in the toolbox rely too heavily on user judgement leading to inconsistency and unrepeatability of the results. It is believed that these issues do not relate to the prediction of the time to failure, which is of most interest to this project.

The method for estimating the time to failure using the toolbox is a simplified approach based on geotechnical behaviour assuming that the failure has initiated and will progress uninterrupted to breach (i.e. the critical shear stress has been exceeded and remains so throughout the duration). The method only considers two values of hydraulic gradient (0.2 and 0.5). Thus, the times predicted by the toolbox may be unrealistic in many situations. For example for some low dams it may be impossible to develop a sufficient hydraulic gradient to initiate failure and for modern dams, with properly designed filters, failure is unlikely to progress to breach even if it did initiate. In other situations, internal erosion may only progress intermittently when the reservoir water level is above a certain threshold.

All these factors need to be taken into account before simply linking the outcome of the toolbox to the required drawdown capacity. Despite its obvious weaknesses, the toolbox is one of the few published methods of estimating time to failure and because it is relatively simple and only needs commonly available parameters it lends itself to application within the guide.

The steps in the process are summarised in Table 8.9 with extracts from the toolbox reproduced in Table 8.10.

Table 8.9 Steps to predicting time to failure using the Internal Erosion Toolbox

Step	Comments	Required data
1. Ability of a soil to support a roof (Table 11.1)	This assesses the dam material's ability to support a roof of a pipe.	Soil classification (including percentage fines and plasticity of fines) and moisture condition.
2. Rate of core erosion (Table 12.2)	Erosion rate index is used; if this data is not available a method for estimating this is given within the toolbox (based on soil classification, percentage fines and liquid limit).	Soil classification with percentage fines or liquid limit, erosion rate index (see comment) and gradient along pipe (options of 0.2 or 0.5).
3. Probability that flow in the developing pipe will be restricted (Table 11.3)	Assess the likelihood that the flow will be restricted.	Details of upstream zones including percentage cohesive fines.
4. Influence of downstream embankment material on the likely breach time (Table 12.3)	The material in the downstream embankment zone's effect on the likely time for a development of a breach.	Material description (e.g. coarse grained rockfill, high plasticity soil).
5. Consolidation of results (Table 12.1)	Based on the previous steps a qualitative and an approximate breach time can be produced. The breach time is expressed as an order of magnitude (e.g. hours, days, weeks or months).	Results from steps 1–4.

Table 8.10 Extracts from Internal Erosion Toolbox (USBR 2008)

Table 12.2 - Rate of Erosion of the core or soil in the foundation

Soil Classification	Best Estimate Erosion Rate Index (I_{HET})	Time for erosion in the core of the embankment or in the foundation	
		Gradient along pipe 0.2	Gradient along pipe 0.5
SM with <30% fines	<2	Very Rapid	Very Rapid
SM with > 30% fines	2 to 3	Very Rapid	Very Rapid
SC with < 30% fines	2 to 3	Very Rapid	Very Rapid
SC with >40% fines	3	Rapid	Very Rapid
ML	2 to 3	Very Rapid to Rapid	Very Rapid
CL-ML	3	Rapid	Very Rapid
CL	3 to 4	Rapid	Very Rapid to Rapid
CL-CH	4	Rapid	Rapid
MH	3 to 4	Rapid	Very Rapid to Rapid
CH with Liquid Limit <65%	4	Rapid to Medium	Rapid
CH with Liquid Limit > 65%	5	Medium to Slow	Medium

Table 12.3 – Influence of the material in the downstream zone of the embankment on the likely time for development of a breach.

Material Description	Likely Breach Time
Coarse grained rockfill	Slow – medium
Soil of high plasticity (plasticity index > 50%) and high clay size content including clayey gravels	Medium – rapid
Soil of low plasticity (plasticity index < 35%) and low clay size content, all poorly compacted soils, silty sandy gravels	Rapid – very rapid
Sand, silty sand, silt	Very rapid

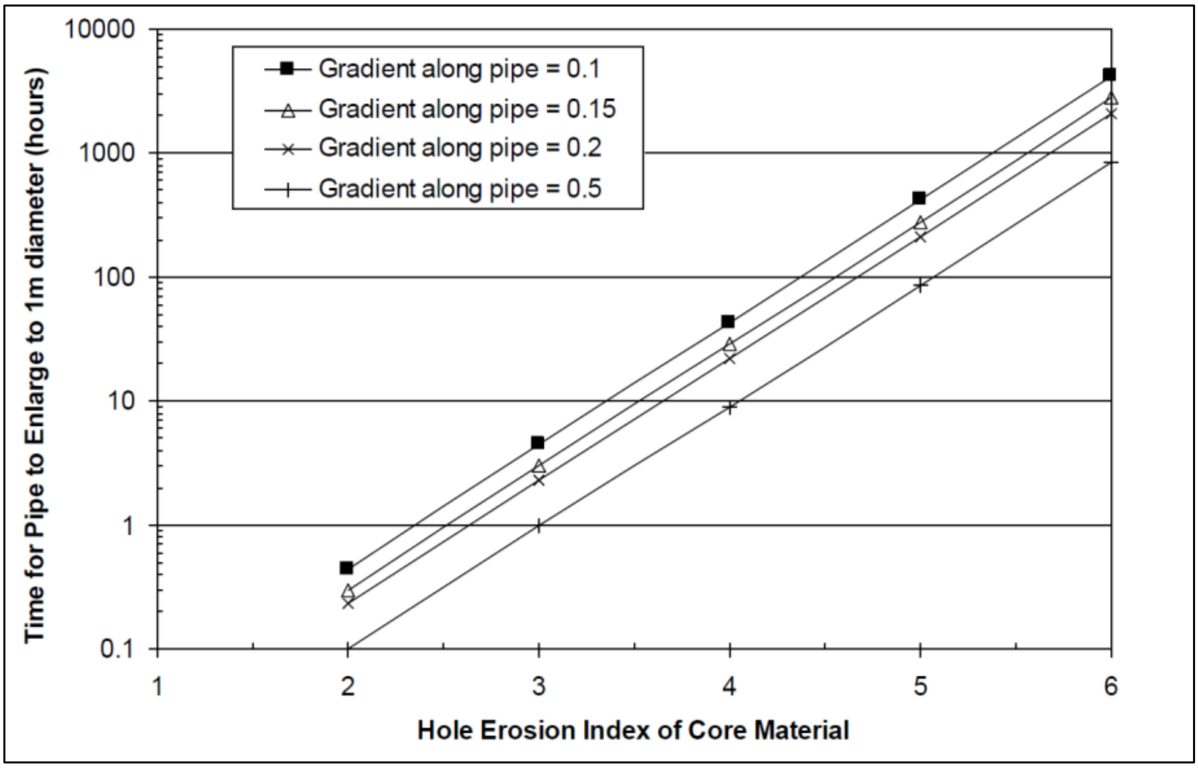
Table 12.4 – Qualitative terms for times of development of internal erosion, piping and breach (Fell et al 2001, 2003).

Qualitative Term	Equivalent Time
Slow (S)	Weeks or months, even years
Medium (M)	Days or weeks
Rapid (R)	Hours (> 12 hours) or days
Very Rapid (VR)	< 3 hours

8.3.4 ICOLD Bulletin 164

ICOLD Bulletin 164 on internal erosion (ICOLD 2013) has been developed recently by the ICOLD committee on embankment dams. There is relatively little guidance on the time to failure by internal erosion except for a few paragraphs of text in Section 8.3. This also includes a graph (Figure 8.1 in ICOLD 2013) indicating the theoretical time it would take for a pipe to enlarge from 25mm diameter to 1m diameter depending on the hydraulic gradient and the erosion rate index of the core material. This graph is reproduced in Figure 8.4.

Figure 8.4 Approximate time for pipe to enlarge from 25mm to 1m diameter (reproduced from Figure 8.1, ICOLD 2013)



The bulletin states that the time for the pipe to enlarge to 2m diameter is approximately 20% longer (i.e. the hole size accelerates exponentially over time).

The graph provides a useful tool to rapidly assess the time to failure. An earlier table in the bulletin (Table 3.4, reproduced as Table 8.11 here) gives typical values of erosion rate index for different types of soils (which is similar to Table 12.2 of the Internal Erosion Toolbox, USBR 2008) making the method relatively quick and straightforward to apply. Alternatively erosion rate index could be determined by soil testing or using the equations given above in Section 8.3.2.

Table 8.11 Representative values of erosion rate index versus soil classification (reproduced from Table 3.4, ICOLD 2013)

Unified Soil Classification	Representative Erosion Rate Index (I_{HET})		
	Likely Minimum	Best Estimate	Likely Maximum
SM with < 30% fines	1	<2	2.5
SM with > 30% fines	<2	2 to 3	3.5
SC with < 30% fines	<2	2 to 3	3.5
SC with >30% fines	2	3	4
ML	2	2 to 3	3
CL-ML	2	3	4
CL	3	3 to 4	4.5
CL-CH	3	4	5
MH	3	3 to 4	4.5
CH with Liquid Limit < 65%	3	4	5
CH with Liquid Limit > 65%	4	5	6

Notes: (1) Use best estimate value for best estimate probabilities. Check sensitivity if the outcome is strongly dependent on the results.
(2) For important decisions carry out Hole Erosion Tests, rather than relying on this table which is approximate.

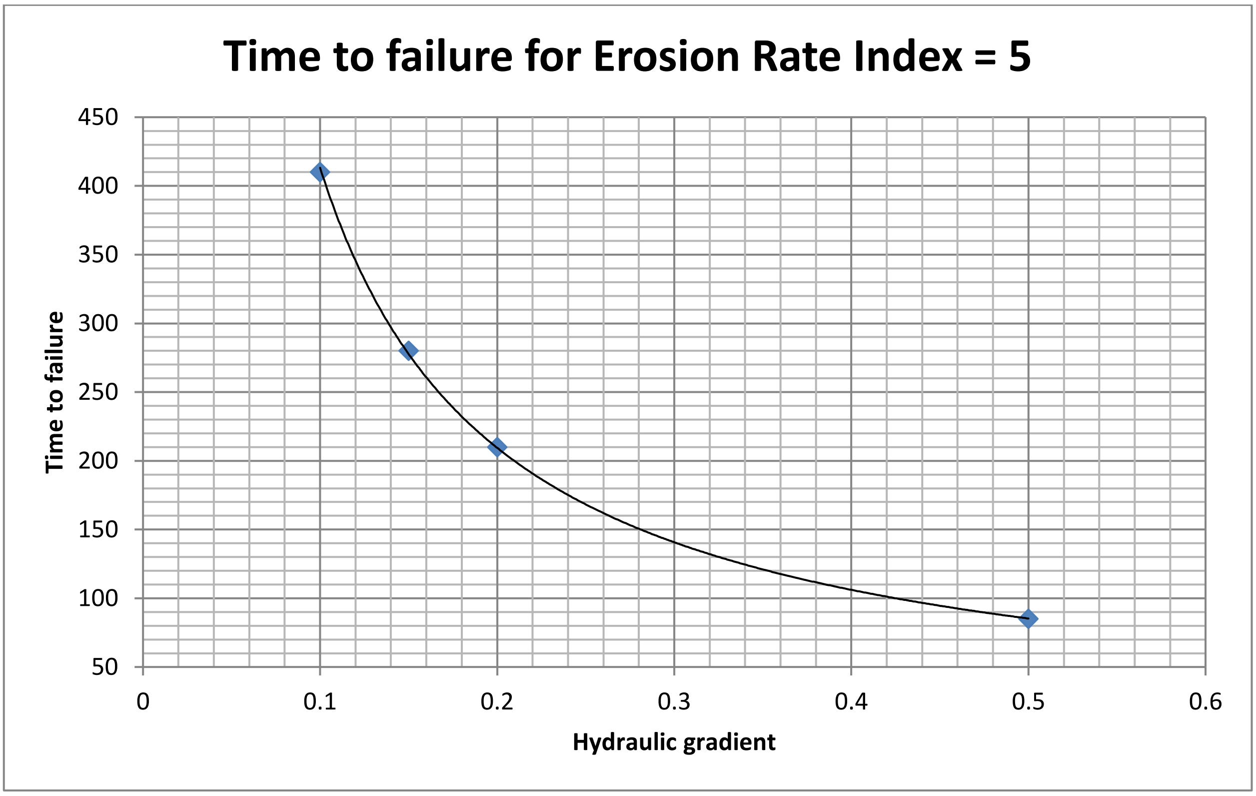
The ICOLD method is considered to offer several advantages over the Internal Erosion Toolbox, namely:

- it covers a wider range of hydraulic gradients
- as part of this project it was possible to recreate the graph from first principles and then create a similar graph for a different range of pipe diameters
- it is arguably simpler and easier to apply, or programme, as part of the guidance
- there is less requirement for user judgement making it more repeatable

There are, however, a number of weaknesses. The time to failure is sensitive to the assumed value of erosion rate index (note the log scale). Also, the time to failure obtained on the y-axis (the time for a hole to enlarge from 25mm to 1m) does not completely match the definition adopted in Section 8.2.3 above. This is discussed in Sections 8.2.4 and 8.2.5 and it has been possible to take this into account by reproducing the graph for an extended range of hole sizes.

For the purposes of the guide, it was considered more useful to present the data as a series of curves showing how time to failure varies with hydraulic gradient for a specific erosion rate index value. An example of such a curve is shown in Figure 8.5. This illustrates the effects of drawing down the reservoir (reducing the hydraulic gradient) on the time to failure. The asymptotic shape of the curve means that the small range of hydraulic gradients covered by the method should be sufficient to assess the time to failure for most UK dams.

Figure 8.5 Alternative method of presenting the relationships in Figure 8.4



8.3.5 Dam breach software models

Overview

Various dam breach software exists which models internal erosion through the progression stage to breach as shown in Table 8.12. Models in the UK have primarily been developed by HR Wallingford with its main model being HR BREACH which was incorporated within the ISIS hydraulic modelling package (InfoWorksRS). HR BREACH has recently been discontinued and is being superseded by two new models:

- EMBREA (**Em**bankment **Brea**ch)
- AREBA (**A** **R**apid **E**mbankment **B**reach **A**ssessment)

Both of these models are still in development stages and were not commercially available at the time of this study (although they have become so subsequently and interested parties should contact HR Wallingford for details). EMBREA models zoned embankments whereas AREBA allows a rapid assessment of homogeneous embankments.

Full details of the theory on which the models are based is not available, which makes it difficult to evaluate the models against the other tools available, although some details for the earlier models (HR BREACH) have been obtained through the software help pages (Innovyze 2013). The software tends to be developed by hydraulic experts with perhaps less emphasis given to modelling the complicated geotechnical issues. For example models often only consider the rate of erosion in a defined diameter hole. It is understood that software has yet to be developed that accurately couples sediment transport theory with hydraulic breach models.

Details of the packages available, and their strengths and weaknesses are summarised in Table 8.12 and described in more detail in the following subsections. The problem with any software model is that it cannot be directly applied as part of the simple guidance document being developed for this project. However, the models can usefully be used to validate the other methods being considered. Due to the lack of details available on the theoretical basis behind the models, and the fact that they are not commercially available, this comparison has been made using output from studies which have used the models. This is presented below.

The option of commissioning HR Wallingford to carry out specific sensitivity analyses using their models for the purpose of this study was considered but was not thought to offer significant value.

Table 8.12 Evolution of software models for dam breach analysis

Software name (and developer)	Approx. date developed	Theoretical basis for erodibility rate (where known)	Parameters required	Comments	Definition of time to failure	Available output from dam breach studies
DAMBRK UK	1988		• Reservoir and valley geometry • Volume depth data • Breach characteristics	This has been used for the majority of breach analyses carried out by UK water companies		None
BOSS DAM BREAK	1999	Does not consider erosion rates	• Breach dimensions • Time to failure	Not used for calculating a time to failure but is designed for assessing the impact of a breach downstream	Time to failure needs to be input into the model	N/A
HR BREACH (HR Wallingford)	2001	Uses Csecs (see Equation 6 in Table 8.7)	• Geometry • Reservoir volume • Erodibility coefficient (Csecs) • Other geotechnical parameters: D_{50}, porosity, dry unit weight, cohesion, plasticity index, friction angle and tensile strength	Previously incorporated as an add-in to ISIS InfoWorksRS software, published by Innovyze No longer supported or developed	Time from continuation to full breakthrough – initial pipe diameter has to be input	The output from two such dam break studies was reviewed (See Table 8.14)
AREBA (HR Wallingford)	2014	Uses the excess stress equation (Equation 1 of Table 8.7) but with an additional empirical coefficient, b, dependent upon soil properties (but often set to 1)	• Geometry (homogeneous only) • Reservoir volume • Erodibility coefficient (this is the only geotechnical property taken into account)	Homogeneous embankments only Strong correlation with results from HR BREACH for homogeneous embankments	Time from continuation to full breakthrough – initial pipe diameter has to be input	*Small reservoirs simplified risk assessment methodology* (Defra/ Environment Agency 2014)
EMBREA (HR Wallingford)	Currently being developed	Believed to be similar to HR BREACH but no details are available	As HR BREACH	Software was not commercially available at the time of this study but is now available	Time from continuation to full breakthrough – initial pipe diameter has to be input	A study using this software was reviewed (Table 8.14)

DAMBRK UK

DAMBRK UK is one of the early dam breach models that uses hydraulic routing instead of hydrologic routing to model downstream affects (Department of the Environment 1991). This has been used for the majority of breach analyses carried out by UK water companies. The breach model requires the breach characteristics to be entered by the user (Binnie and Partners1991a & b).

BOSS DAM BREAK

BOSS DAM BREAK is the simplest model covered in this report and is one-dimensional hydrodynamic flood routing software. The main purpose of this software is to calculate the downstream effects from a resultant flood wave caused by a dam breach and not to calculate the failure time itself. Both piping and overtopping failures can be modelled. Unlike the other models covered in this review BOSS DAM BREAK does not calculate the time to failure of an embankment and instead this has to be input into the model. However, the software help page does give suggested values of breach time for different dam types as shown in Table 8.13. This suggests a time to failure for an embankment dam of between 0.5 and 4 hours, although it is noted that this period is based on overtopping failure.

Table 8.13 Recommended breach parameters for overtopping failure (Boss International help guide)

	Earth Dam	Concrete Gravity Dam	Concrete Arch Dam
Breach Width	1/2 to 4	Some multiple of monolith width	Total dam width
Breach Side Slope	0 to 1	0	Valley wall slope
Failure Time (hrs)	0.5 to 4	0.1 to 0.5	Near instantaneous (≈0.1 hrs)
Pool Failure Elevation	1 to 5 ft. above dam crest	10 to 50 ft. above dam crest	10 to 50 ft. above dam crest

HR BREACH

HR BREACH was developed by HR Wallingford and was implemented in the InfoWorksRS software, published by Innovyze. The method models the formation and growth of a breach by overtopping or internal erosion (piping) taking into account hydraulics, erosion and sediment transport processes, soil mechanics and structural stability. It uses details of the geometry of the dam embankment and its geotechnical properties with the capability of representing both homogeneous and composite embankments where the materials of the core and the outer layers have different properties. The physics of the erosion process is represented by a range of equations suited to different geotechnical properties, in particular soil structure and plasticity index.

As this model is physically based, it offers an improved method for determining the magnitude of a dam break flood over earlier methods, such as DAMBRK UK where the breach formation is usually simulated on the basis of general guidance (Binnie & Partners 1991a), and hence provides an improved basis for decision-making.

HR BREACH takes into account the dam's geometry and a number of commonly available geotechnical parameters such as D_{50}, friction angle and cohesion, but it also takes into account the tensile strength of the material which is less commonly available but is sometimes taken as equal to the value used for cohesion.

Another parameter used for the modelling is 'Csecs'; this parameter dictates the rate a breach will erode back through the dam. This parameter is a critical component for the rate of erosion (see Section 8.3.2).

AREBA

AREBA is the simplest model covered in this report that models the failure of an embankment (unlike BOSS DAM BREAK that uses the failure time as an input to produce outflow). Unlike HR BREACH and EMBREA it can only model homogeneous embankments.

The model takes into account the reservoir's geometry and volume. The only geotechnical parameter required is the erodibility coefficient. Because of these simplifications the model is relatively quick to set up and analyse compared to HR BREACH or EMBREA.

EMBREA

EMBREA is the newest of the dam breach software listed in this report and, at the time of this study, it was only available through HR Wallingford themselves (but it has subsequently become commercially available). The model is somewhat still developmental with HR Wallingford continuing to refine and extend the capabilities of the model. It can model the growth of a breach by overtopping or piping through embankment dams. The model estimates the rate at which an embankment might fail under different hydraulic conditions.

Like HR BREACH, the model takes into account the dam's geometry and a number of commonly available geotechnical parameters such as D_{50}, friction angle and cohesion, as well as an erodibility coefficient.

Review of the time to failure derived by software models

In order to review the predicted time to failure derived using the software, a number of dam breach studies have been obtained where the various models have been used. A summary of the output from these studies is presented in Table 8.14 and compared against the other methods in Section 8.5.

Table 8.14 Summary of results from software models used at a selection of UK dam breach studies

Reservoir(s) **(anonymised at the request of the owners)**	**Study (reference)**	**Model used**	**Dam height**	**Erosion rate index estimate**	**Hydraulic gradient**	**Initial hole diameter**	**Time to failure as indicated by model**	**Comments**
Multiple artificial reservoirs with high, medium and low erodibility	A study into the risks posed by small reservoirs (*Small reservoirs simplified risk assessment methodology*, Defra/Environment Agency 2014)	AREBA	Varies: 2–16m	Assumed as 3, 4 and 5 respectively	0.18 for all cases	N/A see comments	High: 1.6 hours Medium: 8.3 hours Low: Did not fail	**The time to failure is from overtopping. The time for internal erosion failure is not available** Generally small, low erodibility reservoirs did not fail, this is because water discharged out of the reservoir quicker than the breach formation
Reservoir A	HR Wallingford carried out dam breach modelling to provide indicative predictions of the duration of breach formation and the influence of drawdown	EMBREA	13.3m	Plasticity index: 24 Assuming a clay, soil classification: CI Erosion rate index: 4	0.17	10mm	3 hours	Other runs showed a 50mm initial diameter hole led to dam failure in under an hour
Reservoir B	Modelling to assess potential breach scenarios for the proposed reservoir	HR BREACH	22.2m	Embankment properties: PI: 29 (I_{HET}: 4) Greensand foundation properties, D_{50}~1mm	0.1	Not stated	30 hours	Internal erosion through a 3m band of Greensand in the foundation
Reservoir C	Modelling to assess the impacts on downstream reservoir	HR BREACH	2m	PI: 30, LL<65 Soil classification: CH Erosion rate index: 4	0.4	Over-topping failure	7 hours	

8.4 Shortlist of options to predict time to failure

In order to assess the adequacy of the drawdown rate the guide needs to incorporate a simple/rapid method to link to the time to failure for a particular dam, with adjustments for other factors.

Based on the review of relevant literature in Section 8.3, four methods have been identified to estimate the time to failure for an individual reservoir. These methods are summarised in Table 8.15 and their applicability to the guide is evaluated based on the criteria set out in Section 8.1.2.

Table 8.15 Evaluation of options for method to predict time to failure

	Option/ method	Description	Criteria (Note 1)	Score	Comments
1	Internal Erosion Toolbox (USBR 2008)	A defined process following a series of tables (see Section 8.3.3)	Simplicity	MEDIUM	• Uses commonly known parameters
			Repeat-ability	MEDIUM	• Requires some user judgement
			Validity	LOW	• Takes little account of geometry and hydraulic gradient (only 2 options) and therefore is of limited applicability to evaluating the effects of drawdown • Time to failure only given as an order of magnitude
2	Method in ICOLD Bulletin 164 (ICOLD 2013)	A graphical method using erosion rate index. The bulletin also gives typical values for this parameter based on soil classification (see Section 8.3.4)	Simplicity	HIGH	• Uses commonly known parameters • Simple and quick to apply/programme
			Repeat-ability	HIGH	• Less requirement for user judgement making it more repeatable
			Validity	MEDIUM	• Takes into account hydraulic gradient therefore applicable to evaluating the effects of drawdown • Definition of time to failure is different to that suggested (i.e. initial hole size>initial detection size) (but it might be possible to reproduce similar graphs for a different range of hole diameters) • Sensitive to erosion rate index • Does not account for safeguarding features such as filters
3	Software models	Either EMBREA, AREBA or HR BREACH (see Section 8.3.5)	Simplicity	LOW	• Not commercially available (at the time of this study) • Not practical to incorporate into the guide
			Repeat-ability	HIGH	• Quick to run sensitivity analyses
			Validity	MEDIUM	• Accurately models variations in dam cross-section (except AREBA) • Link to hydraulic gradient is buried within the software
4	Expert elicitation	Published expert opinion	Validity	LOW	• Does not allow evaluation with different parameters Only really useful as a tool to compare with other methods
Notes: The methods have been evaluated against the criteria set out in Section 8.1.2. Accuracy is evaluated in Section 8.6.					

Follow-on research could be carried out to investigate the value of dam breach software to carry out sensitivity studies on time to failure, but that is not within the scope of this project. At some point in the future breach modelling software is likely to become more robust and cheap to use, but that is currently still years, if not decades away. Based on this and the evaluation in Table 8.15 there are only two methods (options 1 and 2) that could practically be used within the guide and option 2 offers several advantages over option 1. The results using these methods are compared against the available results from the other methods for a range of typical UK dams in Section 8.5. The data is further validated against actual observed failure times in Section 8.6.

8.5 Application of candidate methods to sample of UK dams

In order to assess whether time to failure is a practical concept for determining an appropriate drawdown rate, and to compare the shortlisted methods identified in Section 8.4, the time to failure has been assessed for a typical range of UK dams using the shortlisted methods.

A sample of typical UK dams has been selected by asking members of the Project Steering Group (and another major water company) to provide embankment cross-sections and geotechnical information for up to three representative dams from their stock of reservoirs. Six dams have then been selected from this group to represent the geographical spread of dam types and geology across England and Wales. The sample also represents a reasonable range of erodibility coefficients from ICOLD Bulletin 164, Table 3.4. The dams chosen are shown in Table 8.16.

The time to failure through internal erosion for each of these reservoirs has been assessed using the Internal Erosion Toolbox and ICOLD methods outlined in Sections 8.3.3 and 8.3.4 respectively. The results of these are shown in Table 8.16 with the reservoir names anonymised.

Table 8.16 Comparison of time to failure from shortlisted methods

				Time to failure (hours)	
Reservoir name (anonymised)	**Summary details**	**Erosion rate index**	**Hydraulic gradient** (Note 1)	**Internal Erosion Toolbox method**	**ICOLD method**
Reservoir 1	26.7m earth embankment with thin puddle clay core	3 to 4 (assume 3.5)	0.18	<3	8.8
Reservoir 2	9.8m earth embankment with puddle clay core	3 (Note 2)	0.17	12 to 24	2.8
		4 (Note 2)	0.17		27
		5 (Note 2)	0.17		264
Reservoir 3	8.5m earth embankment	3	0.11	<3	4
Reservoir 4	13.4m earth embankment	4	0.17	<3	28
Reservoir 5	16.2m embankment	5	0.17	12 to 24	240
Reservoir 6	13.4m earth embankment with thin puddle clay core	5	0.17	12 to 24	240

Notes:

1. Due to the limited options for hydraulic gradient in the Internal Erosion Toolbox method, a value of 0.2 was assumed in all cases.
2. In the case of Reservoir 2, three analyses were carried out assuming different erosion rate indices for the ICOLD method: 3 was based on soil properties in the shoulders, 5 was based on soil properties in the core and 4 was taken as an average. The Internal Erosion Toolbox method takes into account the properties of both the core and downstream shoulder.

The results from Table 8.16 have been plotted in the ICOLD Bulletin format (see Figure 8.4), alongside the output from the software models (see Table 8.14) and the ICOLD relationship lines. This is shown in Figure 8.6 with separate graphs for the two hydraulic gradients (i.e. 0.11 and 0.17/8). Different-shaped symbols have been used to represent the different methods as follows:

- triangles = ICOLD method (by definition these align with the lines)
- squares = piping and seepage tool box method
- circles = results from available software models (see Table 8.14)

In addition, the range of timescales predicted by expert elicitation (see Table 8.6) have been marked on the graph.

Given the number of variables a perfect correlation between the results would not be expected but the graphs do at least show some degree of correlation between the various methods.

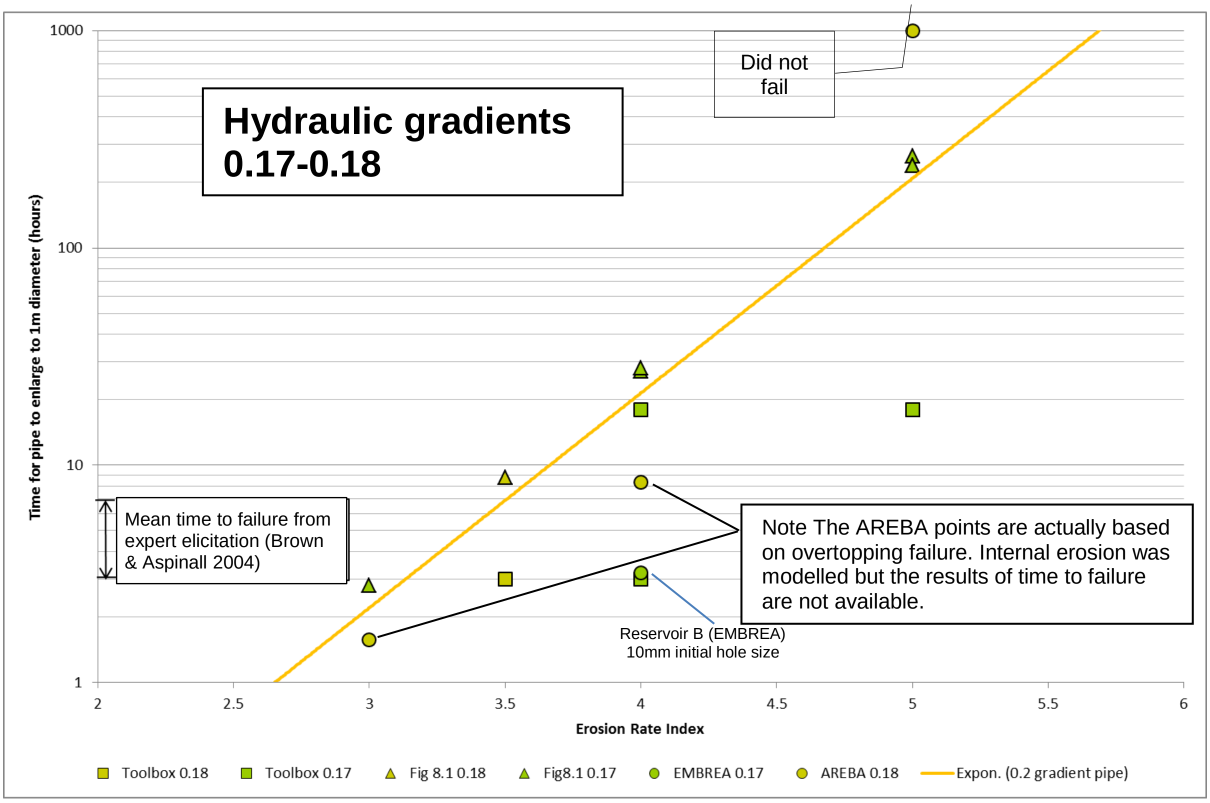
Overall the ICOLD method tends to predict slower times to failure than the Internal Erosion Toolbox method and the software.

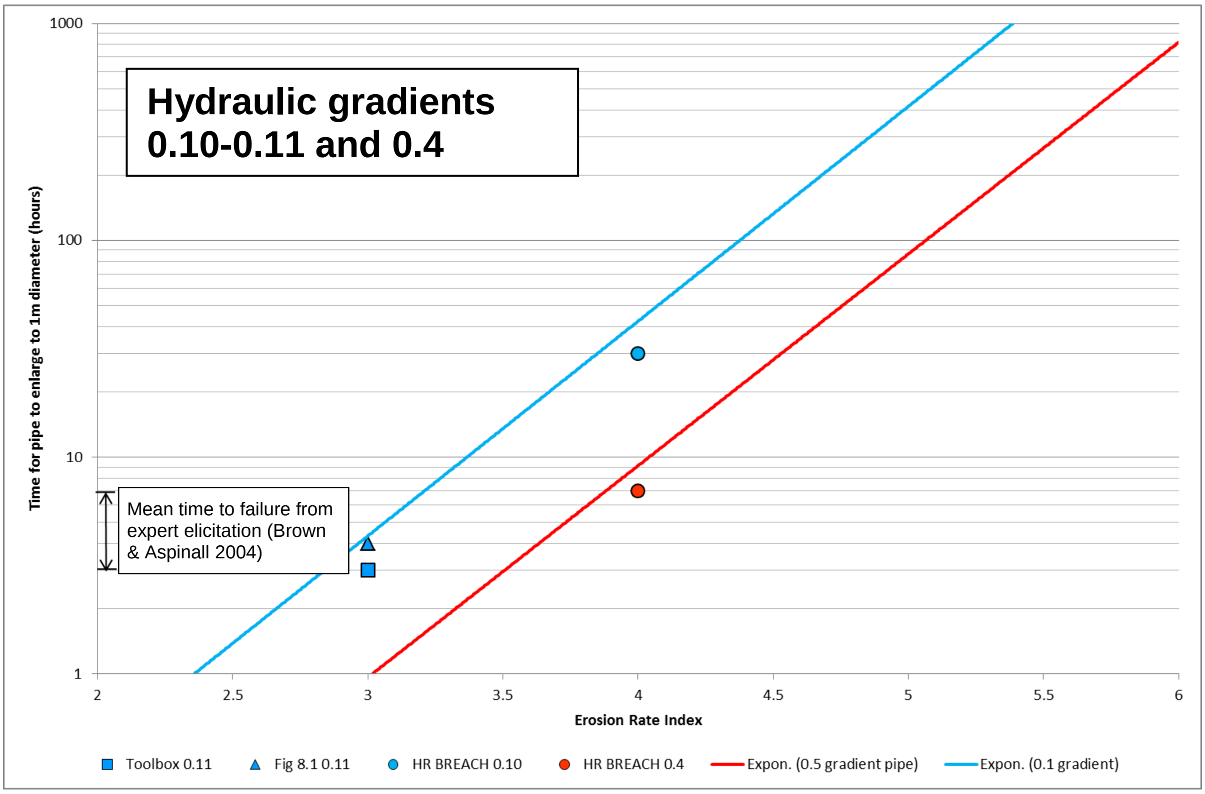
The mean time for reservoir failure identified through expert elicitation (Brown and Aspinall 2004) is shown on the left of the graphs with a time range of 3–7 hours. This compares reasonably well with the other methods for more erodible materials.

Given the huge range of variables and uncertainties in the analysis, overall there is considered to be a sufficient degree of correlation to support adoption of either method. However, due to the advantages offered by the ICOLD method previously discussed, this has been adopted as the basis of the method in Volume 1 of the guide, Appendix D.

A better way to assess accuracy of the methods is to compare them against real failure times observed in actual reported incidents. This is considered in the next section.

Figure 8.6 Time to failure comparison of methods





8.6 Validation of methods against real incidents

The best way to validate the results of the analytical method is to compare them against observations from real-life failure incidents. There are several published sources of incident reports which have been considered as shown in Table 8.17.

Table 8.17 Sources of data reporting on real-life failure incidents

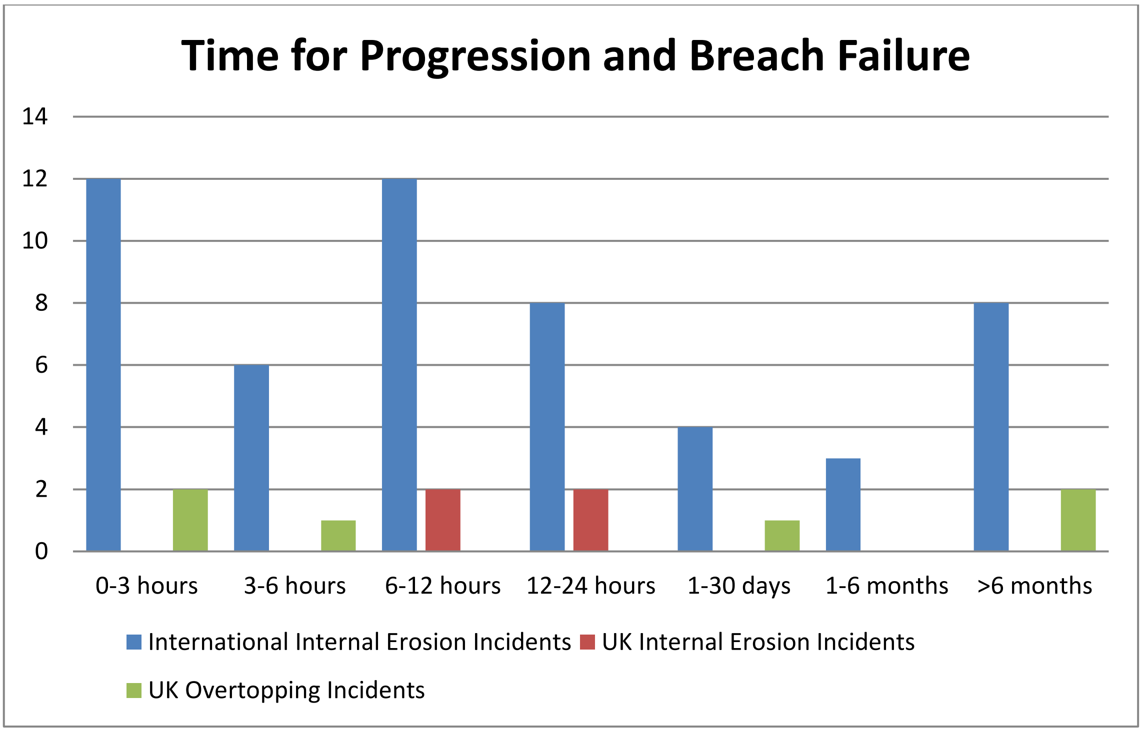
Reference		Description	Comments
1	CIRIA Report 167 (CIRIA 2014)	CIRIA report on lessons from incidents at dams and reservoirs. Lists all the British reservoir failures over the last 200 years	Includes references to other papers with further details of the incidents which have also been researched
2	Charles (2002)	Paper on *Internal erosion in European embankment dams*.	All incidents of UK catastrophic failure already covered by CIRIA
3	Fell et al. (2001)	University of New South Wales report on *The time for development and detectability of internal erosion in piping dams and their foundations*. Tables 3A and 3B give details of 53 incidents (36 through the embankment and 17 through the foundation)	Time to failure is defined as the time from when first concentrated leak is seen to when breach occurs Only two of the cases are from the UK
4	Wickham (1992)	Detailed description of Warmwithens failure in 1970	

CIRIA Report 167 (CIRIA 2014) lists all the British reservoir failures over the last 200 years. It describes 44 incidents of internal erosion failures, eight of which resulted in breach and an indication of timescales is given for six of these. However, details of the time to failure and the geotechnical parameters are not always reported and it has been necessary to make some assumptions in many cases; these assumptions are detailed in Table 8.18). The Fell et al. report (2001) tabulates details of failures from 53 cases largely from overseas. The times to failure from both these references and Wickham (1992) are plotted in Figure 8.7.

Of the four incidents of internal erosion in the UK the time to failure was between 6 to 24 hours.

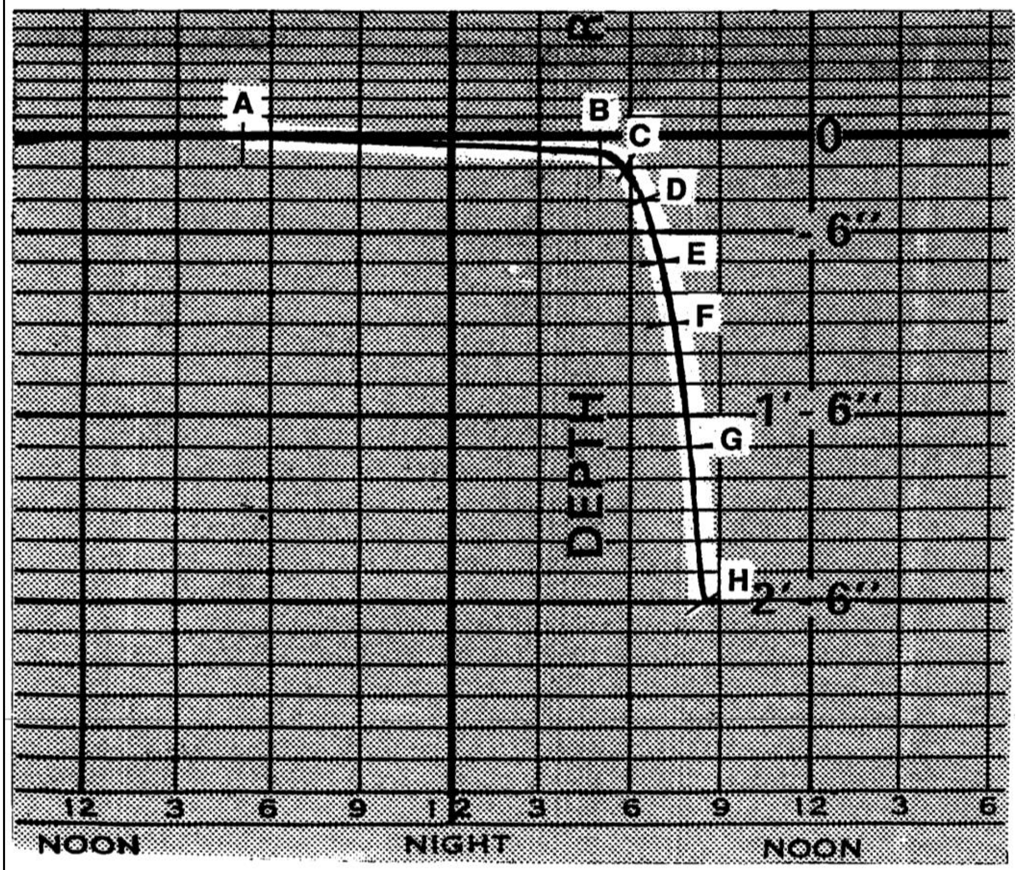
The most detailed records of an internal erosion failure incident are from Warmwithens dam, which failed in 1970 by internal erosion along the interface between a culvert and the embankment (Wickham 1992). The observed reservoir water level during the failure and inferred rates of flow are given in Figure 8.8. Based on our definition of time to failure in Section 8.2.3 (i.e. between points A and C) the time to failure was around 13 hours.

Figure 8.7 Observed times to failure in actual incidents from Wickham (1992), Fell et al. (2001) and CIRIA (2014)



The observed failure times from actual incidents have been compared with the theoretical times predicted using the shortlisted methods in Section 8.4. Only failure incidents in the UK, caused by internal erosion, where the failure resulted in full breach have been considered. On this basis five cases were identified although it was actually only possible to get an indication of time to failure for three of these. Similarly little information is available on the geotechnical properties and assessment has been made on the basis of geological maps. The cases are detailed in Table 8.18 and have been plotted in Figure 8.9 to allow comparison with the prediction methods. For completeness the graph also includes other failures; two where internal erosion occurred for a long time without failure until eventually the reduction in crest level was such that a flood event triggered failure by overtopping and also four overtopping failures.

Figure 8.8 Drawdown of Warmwithens reservoir (Wickham 1992)



Point	Time (hr)	Time Elapsed	Level below Weir (m)	Change in Level (m)	Equivalent Volume (cu.m)	Flow Rate (cu.m/hr)
A	0			0	0	0
B	12	12	0.025	0.025	1020	85
C	12.8	0.8	0.054	0.029	1180	1470
D	13.6	0.8	0.106	0.052	2120	2650
E	14.1	0.5	0.203	0.097	3950	7900
F	14.4	0.3	0.305	0.102	4160	13870
G	15	0.6	0.508	0.203	8270	13780
H	15.5	0.5	0.760	0.252	10270	20540

Notes: 1. Based on our definition of time to failure (i.e. between points A and B/C) the time to failure was around 13 hours

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Table 8.18 Reported incidents of internal erosion failure in the UK resulting in breach

Dam	Failure year	Cause of failure (non-internal erosion shown in grey text)	Time to failure (hours)	Erosion rate index	Hydraulic gradient	Comments and assumptions
Warmwithens, Lancashire 10m high earthfill dam with puddle clay core	1970	Construction of new draw-off tunnel through embankment, providing leakage paths along tunnel	13*	4**	0.16***	*Between points A and C in Figure 8.8. **Based on assumption that Boulder Clay was used for the puddle clay core, which is taken to be low plasticity clay. Since British Geological Survey (BGS) Drift Maps indicate that Boulder Clay is widely present in the area. ***Downstream slope estimated at 1V:3H from photographs. Crest width taken as 3m.
Dale Dyke, Sheffield 29m high earthfill dam (slopes 1:2.5) with puddle clay core (max. core width 4.9m)	1863	Many theories, including leakage from fractured outlet pipe, differential settlement and landslide event	7.5* (may be under-estimated)	4**	7.92***	*Crack was first observed in the late afternoon, and the dam collapsed and breached at 11.30pm the same evening. **Based on nearby borehole logs showing presence of brown clay shale in the area, which is assumed to have formed the puddle clay core (as dam materials were sourced locally). This clay is assumed to have an intermediate to high plasticity. ***Dam consisted of a puddle clay core (3.6m thick) surrounded by relatively permeable shoulder of earth, shale and rock, which are therefore not taken into account.
Horndoyne, Aberdeenshire 5m high homogeneous earthfill dam, impounding 14,000m^3	1990	Leakage along outlet pipe, which eventually formed a small stream	Approx. between 6 and 30*	2.5**	0.16***	*Assumed from limited information: Sources reported that the dam breached 'during the night', and that prior to this a small stream had been noted forming alongside the outlet pipe. The reservoir was filled in 'Late Autumn', and failed on the night of the 17/18 November. **BGS map indicates the presence of till (diamicton) in the area, which is assumed to have been predominantly used for the homogeneous embankment. This is assumed to be a clayey sand for classification purposes. ***Calculated based on maximum pond depth of 5m, and assumed dam slope of 1V:3H. Crest width taken as 2m.
Whinhill, Inverclyde 12m high homogeneous earthfill dam	1835	Animal burrowing, allowing water to erode away embankment	No details available	2.5*	-	* BGS map indicates the presence of till (diamicton) in the area, which is assumed to have been used for the homogeneous earthfill dam. This is taken to be a clayey sand for classification purposes.
Kellington East, Rawcliffe 3m high homogeneous earth embankment on River Aire	2008	Animal burrowing	No details available	4*	-	*Based on description of embankment material as 'compacted clayey material'. BGS maps show presence of alluvium, judged to be intermediate to high plasticity clay.
Blackbrook, Leicestershire 11m high earthfill dam with puddle clay core (1.8m core width)	1799	Overtopping from snow melt, but largely due to crest settlement associated with internal erosion	2 years*	4**	6.1***	*Suffered leaks as soon as it began to fill in 1797 (CIRIA 2014), i.e. internal erosion was probably continuing over this time. **Poor quality clay core, described as riddled (Binnie 1987). Judged to be of a high plasticity (Reeves et al. 2006). ***Outer slopes consisted of small lumps of rock mixed with soil (Binnie 1987).
Bilberry, West Yorkshire 29m high earthfill dam with puddle clay core (max. core width 4.9m)	1852	Overtopping from rainfall event, following 3m of settlement caused by internal erosion	11 years*	4**	5.9***	*First evidence of leakage occurred in 1841, and the crest continued to settle until the dam failure in 1852, i.e. internal erosion was probably continuing over this period. **Borehole logs in the area indicate presence of mottled clay, taken to have an intermediate to high plasticity. ***Based on dimensions from diagrams, 29m height and 4.9m clay core width (CIRIA 2014).
Earlsburn, Stirling 6m high earthfill dam with silty clay core	1839	Earthquake (assumed to be internal erosion)	8 hours*	3.5**	0.13***	*Dam failed approx. 8 hours following earthquake (CIRIA 2014). Assume that internal erosion was initiated by earthquake. **Silty clay central core, classified as low plasticity (Musson 1991). ***Dam dimensions taken from Musson (1991). Shoulders constructed with peat and earth, so whole dam width was considered.

Dam	Failure year	Cause of failure (non-internal erosion shown in grey text)	Time to failure (hours)	Erosion rate index	Hydraulic gradient	Comments and assumptions
Cwm Carne, South Wales 12m high earthfill dam, with puddle clay core	1875	Overtopping	5.5 hours*	4**	No details available	*(CIRIA 2014). **Puddled clay described as poor quality, so assumed to have high plasticity.
Brent, London 7m earthfill dam with 1.8m wide puddle clay core	1841	Breach following thaw of River Thames and 7 days of heavy rainfall	7 days*	4**	No details available	*Assume failure began during thaw and at the onset of heavy rain period. **Puddle clay core, assume intermediate to high plasticity.
Woodhead No.1, Longendale Valley 7.3m high earthfill dam with puddle clay core	1849	Overtopping during construction	3 hours*	4**	No details available	*(CIRIA 2014). **Puddle clay core, assume intermediate to high plasticity.
Trewhitt Lake, Northumberland 5m high homogeneous earthfill dam	1963	Overtopping	20 minutes	3*		* BGS map indicates the presence of till in the area, which is assumed to have been used for the homogeneous earthfill dam. This is taken to be a clayey sand for classification purposes.

Figure 8.9 Comparison of actual observed times to failure against results from methods of prediction

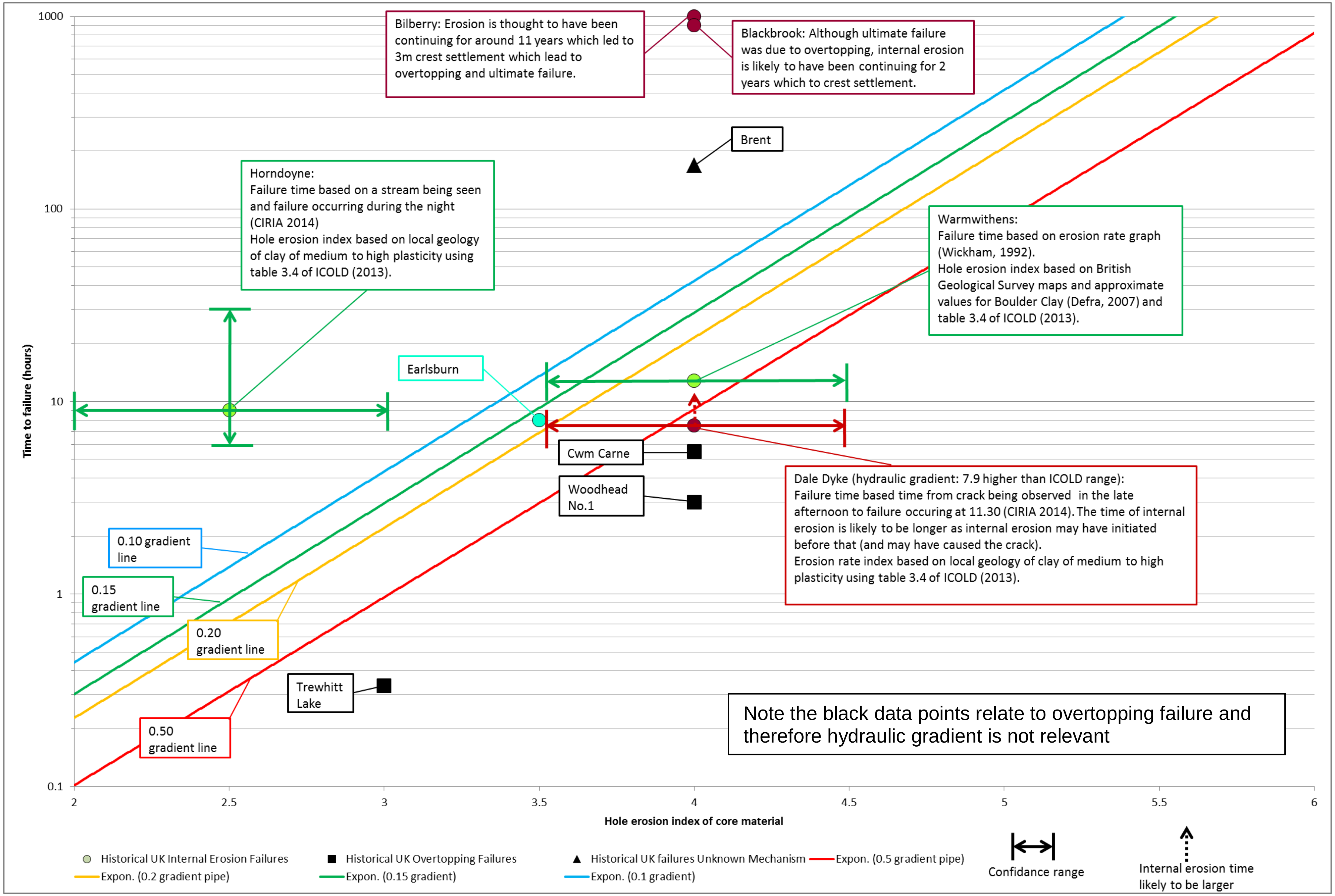


Figure 8.9 is based on Figure 8.4 with the times to failure and approximate erosion rate index from the real-life incidents overlaid on the graph. Colours have been used to indicate the different hydraulic gradients in each case such that, in theory, the coloured points should align with the corresponding coloured lines, or shades between them. The black points relate to incidents of overtopping failure where hydraulic gradient is not relevant. Due to the uncertainties and assumptions relating to these incidents (see Table 8.18), error/confidence bands have also been indicated on the graph.

Dale Dyke

The Dale Dyke failure correlates reasonably well with the ICOLD method. The assumed value of hydraulic gradient of 7.9 which neglects the shoulders has been compared with the highest value of hydraulic gradient from the ICOLD graph of 0.5. It is possible that the assumed failure time of 7.5 hours is underestimated because this is measured from the time a crack was first observed and signs of initial internal erosion might have been detectable earlier.

There is no reliable data on the erosion rate index of the material and as previously stated the ICOLD method is sensitive to this parameter. A best estimate erosion rate index of 4 has been assumed based on the fact that the dam was constructed from locally won materials and the BGS viewer describes nearby material as 'brown clay shale' which would typically be intermediate to high plasticity. An error band has been shown on Figure 8.9 to account for this.

Warmwithens

The time to failure and hydraulic gradient at Warmwithens are better understood but again there is some uncertainty over the erosion rate index. The best estimate erosion rate index of 4 has been based on the assumption that Boulder Clay is widely present in the area (as determined from the BGS viewer) and it is likely that this low to intermediate plasticity clay was used for the dam construction. On this basis, the ICOLD method would have predicted a slower failure time than was actually observed.

Horndoyne

This incident relates to a small 5m high dam not covered by the Reservoirs Act 1975. Unlike the other two cases, the observed time to failure in this incident was slower than might be predicted using the ICOLD method. Again there is some uncertainty in the erosion rate index, and also the actual time it took to fail.

Earlsburn

This correlates reasonably well with the ICOLD method, although the actual observed failure time is a couple of hours slower than the ICOLD method would predict if the assumed parameters are correct.

Bilberry/Blackbrook

It is believed that internal erosion may have been continuing for a period of years in both these incidents before failure actually occurred. In that case they do not agree with the ICOLD method which would predict a much faster failure.

Overall

Overall there is not enough published data to allow a proper validation to be carried out and the quality of the data is limited. The uncertainties in the observed data, particularly relating to the erosion rate index, make precise validation of the prediction method difficult. However, the exercise does indicate that the ICOLD method predicts times to failure within a reasonable range of observed times in most of the incidents considered. The ICOLD method has therefore been adopted as the basis of the method in Volume 1 of the guide, Appendix D, although the method was refined to be based on an initial hole diameter of 5mm rather than 25mm. The effect of this refinement can be seen by comparing Figures 8.10 and 8.11. As shown in Figure 8.10 the spreadsheet model, which was developed from first principles, replicated Figure 8.1 of the ICOLD Bulletin reasonably well.

Figure 8.10 Spreadsheet model developed during the project to replicate ICOLD Figure 8.1

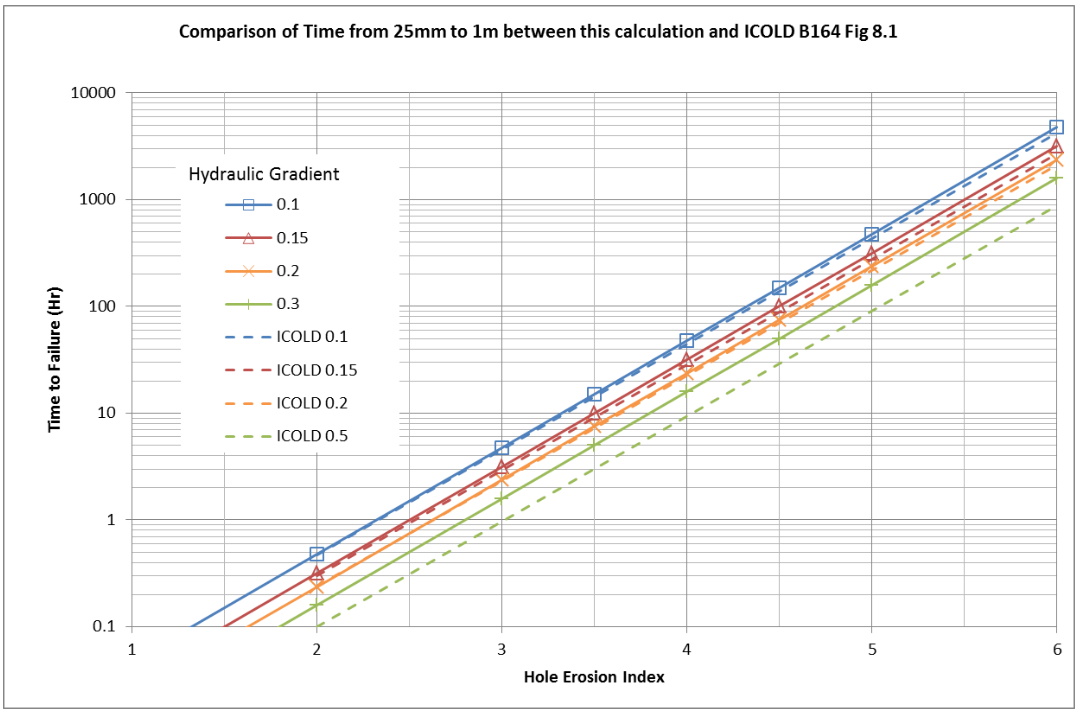
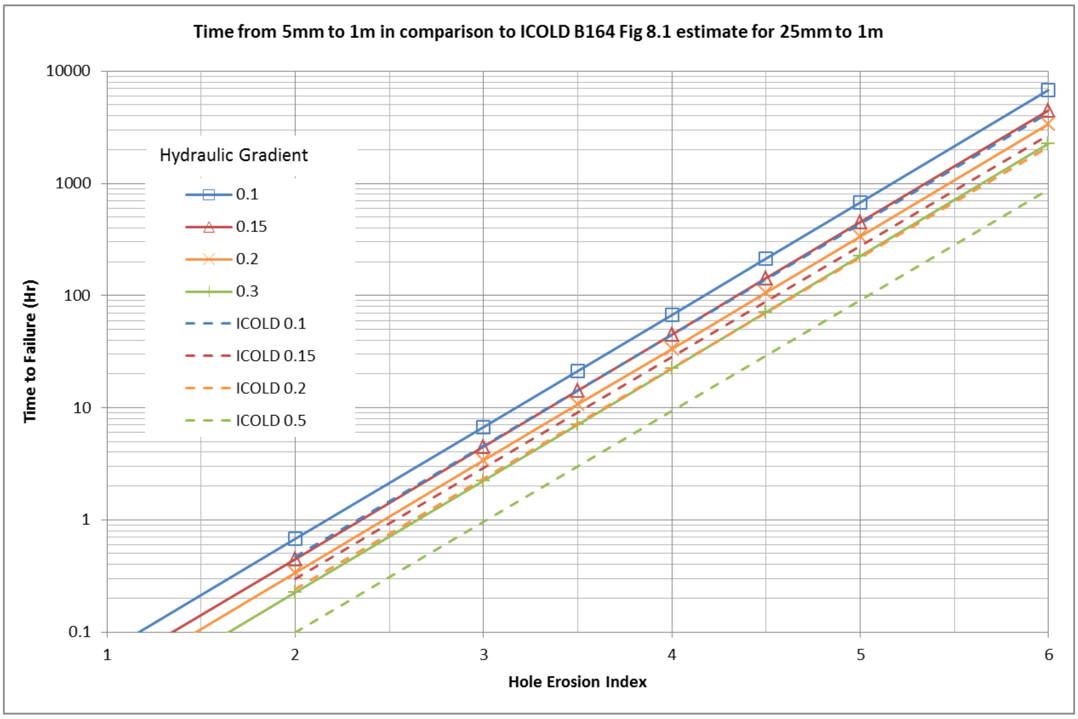


Figure 8.11 Extension of spreadsheet model to consider initial hole size of 5mm, compared with ICOLD relationship which is based on a 25mm initial hole size



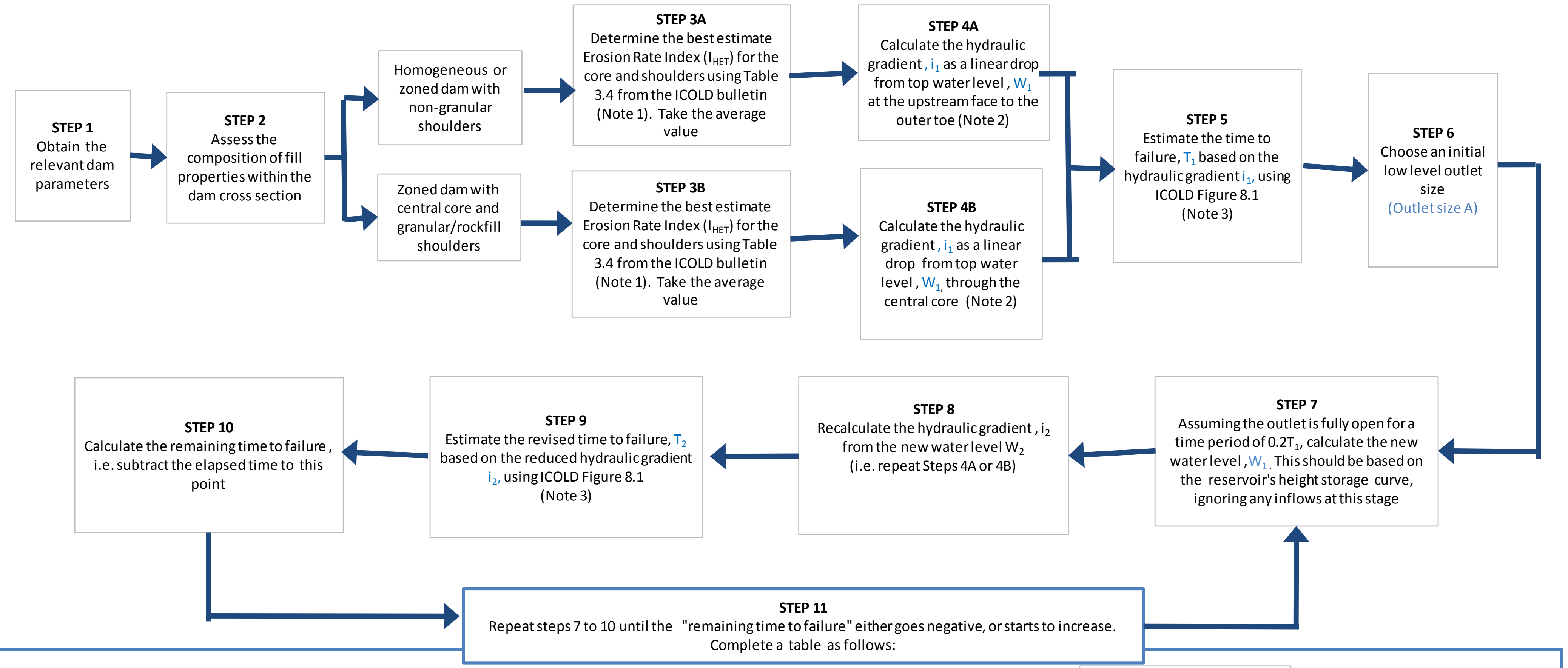
The method was used to determine theoretical drawdown rates required to avert internal erosion as discussed in the next section.

8.7 Calculation of theoretical drawdown rates required to avert internal erosion

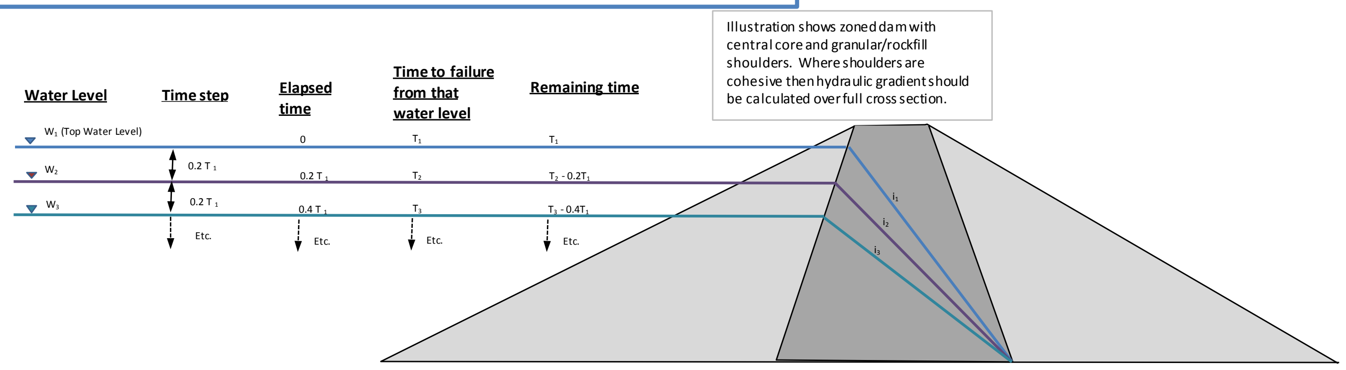
As discussed in the previous sections the guidance links drawdown capacity to the time it would take a particular dam to fail through internal erosion. In order to calculate this time the method given in ICOLD Bulletin 164 (ICOLD 2013) has been found to be the preferred method because it is simple and rapid to carry out, has a strong link to hydraulic gradient (which is important when assessing the effects of drawdown) and the results broadly agree with actual observed failure incidents.

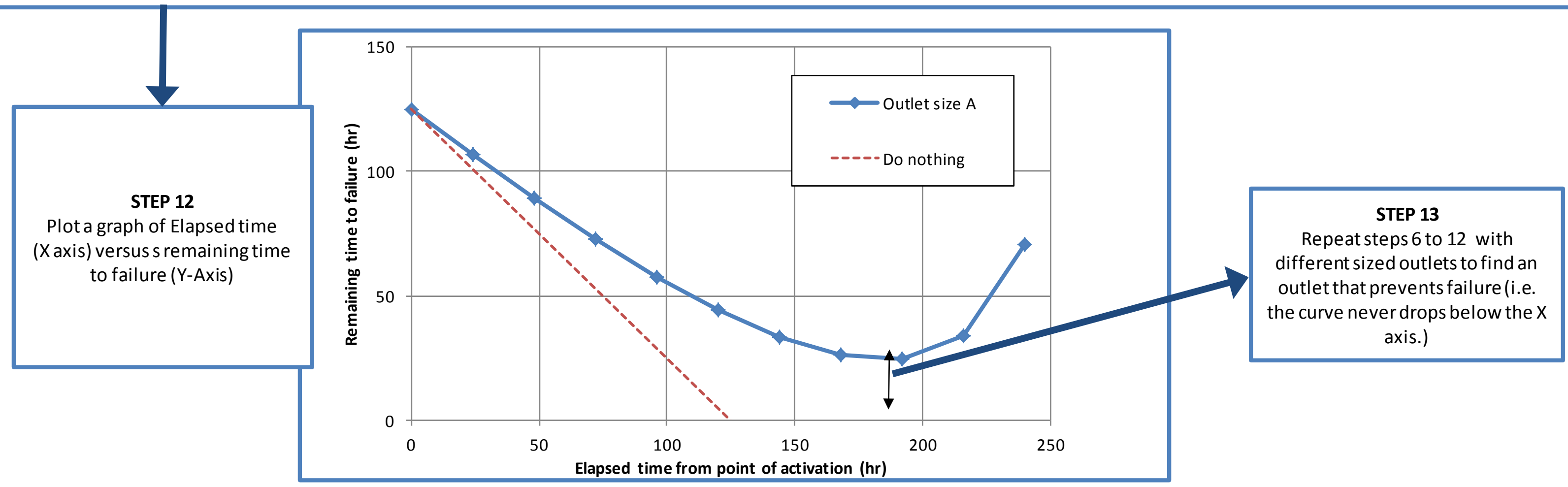
Figure 8.12 shows a system incorporating this method which was initially proposed for use in the guide to determine a recommended minimum outlet size. The approach was refined since this flow chart was developed but the fundamental principles were retained to derive the data points on Figure D.1 in Volume 1 of the guide. Further details of how Figure D.1 was derived are explained in the 'Information box' provided with the graph.

Figure 8.12 Early version of the system proposed to determine a minimum required outlet size (the approach was subsequently refined but the fundamental principles were retained)



Outlet size A				
Elaped time	Water level in reservoir	Hydraulic gradient	Time to failure with that hydraulic gradient	Remaining time to failure (i.e. subtract elapsed time to this point)
0	V_1	i_1	T_1	T_1
$0.2\,T_1$	V_2	i_2	T_2	$T_2 - 0.2\,T_1$
$0.4\,T_1$	V_3	i_3	T_3	$T_3 - 0.4\,T_1$
$0.6\,T_1$	V_4	i_4	T_4	$T_4 - 0.6\,T_1$
$0.8\,T_1$	V_5	i_5	T_5	$T_5 - 0.2\,T_1$
$1.0\,T_1$	V_6	i_6	T_6	$T_6 - 1.0\,T_1$





Notes:

1. Alternatively the erosion rate index could be determined by directly measuring it in laboratory tests or derived by equations based on dry density and percentage clay. Further advice is also given in Defra (2007), Appendix C.
2. The system shown is based on internal erosion occurring at the base of the dam. A similar system could be developed to consider failure through the dam at a higher level.
3. When this flow chart was developed it was anticipated that the guide would include a series of graphs to allow users to look up the time to failure based on hydraulic gradient (i.e. similar to Figure 8.5). In practice a series of curves similar to Figure 8.5 were derived for different erosion rate index values and these were used in a spreadsheet model.
4. This flow chart does not take into account other factors such as concurrent inflows, frequency of surveillance or presence of filters. The factors are discussed in Volume 1 of the guide.
5. This system includes several simplifications and approximations including:

- No account of critical shear stress is made (see Volume 1, Appendix D.2). This means that the theoretical drawdown rates may be **overestimated.**
- It does not take into account the fact that the size of the erosion hole is bigger at the start of each new time step than it was for the previous iteration. Instead, the time to failure at each time step, has been taken as the time for a hole to develop from 5mm to 1,000mm. This approximation means that the theoretical drawdown rates are **underestimated.**
- Flow out of the leak was conservatively neglected when calculating the falling head, because it would be illogical for the guide to allow uncontrolled leakage to be considered a benefit. This means that the theoretical drawdown rates may be **overestimated**.

It is considered that the above three approximations will broadly cancel each other out and thus the system is deemed appropriate for gaining a rough indication of the theoretical rate required to avert failure, but as with any theoretical model of this type, the results should be considered within an overall framework of engineering judgement.

The general system illustrated in Figure 8.12 was refined to determine the minimum drawdown capacity required to avert failure (i.e. to ensure the curve in Step 13 never drops below the X axis). The system relies on the following parameters:

- dam height
- erosion rate index
- hydraulic gradient
- reservoir depth–storage relationship

For each of these parameters, the likely range of values typically found in the UK were identified and the system was applied for various permutations of the maximum and minimum values of each parameter to determine the theoretical drawdown capacity required to avert failure.

It was possible to eliminate dam height as a variable by expressing drawdown rate as a percentage of the maximum reservoir depth per day. This allowed the theoretical drawdown rate to be plotted against erosion rate index and hydraulic gradient in Volume 1 of the guide, Figure D.1, assuming a standard V-shaped valley where the storage capacity increases exponentially with height.

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Appendix A Summary of literature review

A1 Summary of literature review (see seperate file)

A2 Review of severity 1 incidents (sudden uncontrolled release of water) from CIRIA Report SP167 (CIRIA 2014) (see seperate file)

Appendix B Data collected from Environment Agency's database of Section 10 reports

Appendix B1	Information extracted from review of 197 inspection reports held on the Environment Agency's database (see seprate file)
Appendix B2	Drawdown criteria used by UK inspecting engineers extracted from Section 10 reports

Table B2 Drawdown criteria used by UK Inspecting Engineers extracted from S10 Reports (2004-2014)

Ref	Criterion	% res height/day	Inflows	Source	Reservoir/IE
1	1m/day	18.9	Not stated (non-impounding)	Not stated	Gilstead (Yorkshire Water), Andy Hughes
2	1m/day	10	No inflow (non-impounding)	Not stated	**King George V (Thames Water), Andy Hughes**
3	320mm/day	9.1	Not stated	Not stated	Cheveney Farm Upper Lake (Cheveney Farm). Mike Headling
4	449mm/day	7.7	Not stated	Hinks' rule	Earlswood Lakes (Canal & River Trust), Jonathan Hinks
5	1m/day	7.1	No Inflow	United Utilities plc	Springs
6	1m/day	7.1	No inflow	Not stated	Springs (United Utilities), Andy Hughes
7	500mm/day for Category C	7.1	Not stated	Not stated	Westbeck Lake (Warter Priory Farms), Douglas Gallacher
8	300mm/day	6	Not stated	Not stated	**Black Lake (Prestwood), Martin Airey**
9	300mm/day	5	No inflow	Not stated	Langold Lake (Bassetlaw DC), Alan Warren
10	300mm/day	5	Not stated	Not stated	Blackleach (Salford CC), Martin Airey
11	500mm/day	4.5	Not stated	Not stated	Lawton Hall Lake(Haddon), Keith Gardiner
12	420mm/day	4.4	Not stated	Hinks, 2008 (reference is also made to CRT approach)	Bretton Park Lake (Wakefield MBC), Jim Claydon
13	1m/day for Category A	4.3	Not stated	Not stated	Tittesworth (Severn Trent), Brian Charles Morris
14	50% volume within 5 days	4.1	Winter daily mean inflow	Canal and River Trust	Barrowford (Canal & River Trust), John Gosden
15	50% volume within 5 days	4.1	Winter daily mean inflow	Canal and River Trust	Rishton (Canal & River Trust), John Gosden

Ref	Criterion	% res height/day	Inflows	Source	Reservoir/IE
16	300mm/day	3.8	Q10	Not stated (reference is also made to CRT approach)	Denton (Canal & River Trust), Tim Hill
17	500mm/day	3.3	100l/s	Not stated (understood to be Hinks, 2009)	**Sutton Bingham (Wessex Water), Jonathan Hinks**
18	3% height/day	3	Q10	Scottish Water	Gartmorn Reservoir
19	3% height/day	3	Q10	Scottish Water	Loch Goin Reservoir
20	400mm/day	2.4	Not stated	Hinks' rule Dams and Reservoirs, 2009	**Warland (United Utilities), Jonathan Hinks**
21	50% of volume in 10 days	2.1	No inflow	Anglian Water	**Pitsford (Anglian Water), Andy Rowland**
22	50% of volume in 10 days	2.1	Not stated	Anglian Water	**Alton Water (Anglian Water), Ian Carter**
23	Draw the reservoir down to 70% depth in about six days	1.9	Not stated	Draft Interim Guide to Emergency Planning, May 2006	Dronfield Dam (Environment Agency), Ian Gowans
24	50% volume in 20 days, emptying in 30 days	1.03	No Inflow	Not stated	**Graffham Water (Anglian Water), Ian Carter**
25	25% of reservoir volume in 28days	0.33	Not stated	Northumbrian Water Ltd	Grassholme
26	50% volume – duration not stated	-	Not stated	Canal and River Trust	Welford (Canal & River Trust), Andy Rowland

Bold=MIOS for outlet from Alan Warren's report (Warren 2012)

Appendix C Responses from major dam owners

JACOBS

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Appendix C1 Letter to Water Companies

21 July 2014

Name
Company Name
Address Line 1
Address Line 2
CITY, Post Code, UK

Our Ref: B228N601/JDG/001

Determining the optimum hydraulic capacity of low level outlets for reservoirs

Dear xxx

In the UK there is currently no single accepted approach to determining what represents an acceptable reservoir drawdown capacity. Some reservoirs owners adopt their own internal guidance standards while some Panel Engineers apply their own rules of thumb when advising on drawdown capacity as part of a Section 10 Inspection under the Reservoirs Act 1975.

We have been commissioned to carry out a research project and produce industry guidance on the optimum capacity of low level reservoir outlets. The primary aim of this project is to provide a measure of consistency to the assessment of the minimum drawdown capacity at any particular reservoir. The study is part of the joint Environment Agency / DEFRA Flood & Coastal Risk Management (FCRM) research and development programme.

We are currently in the initial stages of the project and are reviewing the existing approaches applied to reservoir outlet design, in the UK and internationally. As the Reservoir Safety Manager for XXXX water company we would be extremely interested to understand the rationale which you currently adopt for sizing reservoir outlet capacity. For example:

- Do you have a standard minimum reservoir drawdown rate which you apply
- Do you allow for any reservoir inflows when calculating the drawdown rate, in which case how much?
- How much reliance do you accept on mobile pumps and siphons etc. compared to fixed outlets?

Answers to these questions, or other information you can provide us with, would be very much appreciated and will assist in developing standard UK guidance for the benefit of the industry.

As the project develops we may also wish to contact you later in the project with further requests for information or a more comprehensive questionnaire and would be grateful if you would indicate your willingness to be contacted further.

Yours sincerely

John Gosden
Jacobs All Reservoirs Panel Engineer

Table C2 Summary of responses from major UK dam owners

Organisation	**Drawdown rate**	**Criterion expressed in approx. % reservoir height / day (note 1)**	**Inflows**	**Permanent installation of temporary pumps**	**Other**
Thames Water	1m/day at TWL; recommendations of Panel in 2000, vulnerable to rapid failure, high consequence, quick reduction of 10% of water height likely to have significant impact on erodibility; surveillance around 2 or 3 times per week	10	None as non-impounding and can turn off pumps	No pumps considered; large surface areas mean capacity to affect drawdown requires an unfeasibly large number of pumps (both provision and fuelling). Could be possible as back-up at smaller reservoirs if permanent provision failed. Do include discharge through wtw but only if a %age of max output (that which wtw output has always met) and phasing this out as not 100% reliable	Service reservoirs; no policy but new reservoirs require a drain to empty one cell in 12 hours. At certain times of day customer demand can drop level quite rapidly. Flood storage reservoirs; no provision is possible or necessary
United Utilities	1m/day	7.7	No but do undertake sensitivity analysis to baseflow and partial valve opening to assess need for upgrade or possibility of deploying pumps	Contingency plans do include how much additional capability can be provided at each site by mobilising pumps. In-house pumping capability of 100Ml/day, which can be supplemented by external suppliers	
Canal & River Trust (CRT)	Reduce the impounded volume to 50% of the reservoir's maximum capacity in the following timescales: Category A1 Reservoirs: 5 days with reservoir surveillance once a week, or 3 days with reservoir surveillance twice a week. Category A2 Reservoirs: 7 days with reservoir surveillance once a week, or 5 days with reservoir surveillance twice a week. Category B,C and D Reservoirs: 9 days with reservoir surveillance once a week, or 7 days with reservoir surveillance twice a week.	*2.3-4.1*	The inflow to the reservoir during the drawdown period is the average winter daily flow from the reservoir catchment (except where bywash channels or inflow diversion arrangements have the capability of reducing the flow).	The National Framework Contractor will supply any pumping equipment required together with the resources for its deployment. CRT remains in control of the operation acting as 'main contractor' with the framework contractor as subcontractor. *Paper by Brown 2009 says may be augmented with up to 1m3/s by temporary pumps*	

Organisation	Drawdown rate	Criterion expressed in approx. % reservoir height / day (note 1)	Inflows	Permanent installation of temporary pumps	Other
Scottish Water	*No response to date. Paper by Mann 2014 suggests they use Hinks' rule for first 24 hours then CRT rule but with relaxations for specific aspects) (see Section 3.1.2)*		*No response to date. Paper by Mann 2014 suggests Q10 flows*	*No response to date. Paper by Mann 2014 suggests pump capacity equal to the capacity of the existing permanent installation has been adopted for "further drawdown", up to a practical maximum installed of 1.5m³/s, and one day per 0.5m³/s capacity is allowed for mobilisation and installation*	
Welsh Water	*No specific standard used and adequacy is determined by Inspecting Engineers within Section 10 Reports*	*N/A*	*Dependent on the Inspecting Engineer but is often not taken into account.* *Base flow is assumed when determining the maximum allowable drawdown rates.*	*Welsh Water have bought two high capacity pumps with a capacity of 0.3m³/s, although they do not rely on these.*	
Severn Trent Water	*Most conservative out of Hinks' rule(i), Drawdown to 50% loading in 14 days for Cat A/B (ii) and 30 days for Cat C/D(iii).*	*(i)2.7 (Hinks)* *(ii) A/B: 1.8* *(iii) C/D: 0.8*	*Q10 for Hinks*	*consider the use of emergency pumps and the use of the Fire service HVPs (not large capacity but quick response)*	*One of their sites has a permanent syphon*
SSE	*No specific standard used and adequacy is determined by Inspecting Engineers within Section 10 Reports*	*N/A*	*Dependent on the Inspecting Engineer, generally average inflow or baseflow has been used.*	*Generally do not reply on pumps, scour capacities are often much greater than the amount of pumping capacity you can realistically bring to site.*	
Northumbrian Water	*No response to date. Paper by Prentice 2005 suggests their standard is to drawdown to **25%** capacity in **28 days***	*1.3*	*No response to date. Paper by Prentice 2005 suggests Winter 28 day peak*		

Organisation	Drawdown rate	Criterion expressed in approx. % reservoir height / day (note 1)	Inflows	Permanent installation of temporary pumps	Other
Anglian Water	*No response to date. Paper by Tam 2012 suggests their standard is Drawdown to **50%** capacity in **10 days** (20 days for non-impounding/small relative catchment).*	*2.1 (1.0)*	*No response to date. Paper by Tam 2012 suggests 0.5m^3/s*		
Wessex Water	*No response to date. Paper by Wellbank 2008 suggests their rule is to draw*down to 75% level within approx. 3 days	3.0			
South West Water	*No response to date*				
Bristol Water	No standard; using 1m/day for new planned reservoir (Cheddar 2)	6.5% (Based on Cheddar 2 being 15.5m high)	No standard; suggest average annual inflow	Only permanent can be considered reliable	
Yorkshire Water	*No response to date*				
Northern Ireland Water	*No response to date*				

Notes:

1. Where the criterion applies to a specific dam or a specific water company's stock of dams then the daily drawdown criterion has been expressed as a percentage of the dam height or median dam height. The median dam height for each company's stock of dams has been determined from the BRE Database of UK dams, 2004. Where the criterion is defined in terms of reservoir volume, it is assumed that 50% reservoir volume equates to 79% reservoir height and 25%volumen equates to 91% reservoir height (assuming a simplified cone shaped reservoir basin)
2. When a rate has been given for multiple reservoirs the mean reservoir height has been used to calculate the %reservoir height / day.

Appendix D Industry questionnaire

Appendix D1	UK questionnaire to registered reservoir owners and BDS members
Appendix D2	International questionnaire
Appendix D3	Supplementary guidance for completing the questionnaire

Industry survey questionnaire

Please read the guidance note before filling out this questionnaire, it will help you respond to the questions and also inform you of any questions you do not need to complete. If there is insufficient space for your answer please continue on an additional sheet.

	Name:	…………………………………………………………………………………………
	Organisation:	…………………………………………………………………………………………
Q1.	Does your organisation, or do you personally, own or operate any reservoirs? ☐ Yes (owned personally) ☐ No (go to Q4) ☐ Yes (through my organisation) Number of reservoirs operated ……………………	
Q2.	Of the reservoirs referred to in question 1 how many: Have a functioning permanent emergency outlet/ scour outlet: ………………………………… Have a non-functioning permanent emergency/scour outlet? ………………………………… Have no permanent emergency outlet/ scour outlet: …………………………………	
Q3.	How often are your reservoirs visited by a responsible person: Frequency / (Please tick) / Comments on which of your reservoirs this applies to At least once every 3 days ☐ ……………………………… Between 4 days and 1 week ☐ ……………………………… Between 8 days and 2 weeks ☐ ……………………………… Less often than once every 2 weeks ☐ ………………………………	
Q4.	Are you a Panel Engineer appointed under the Reservoirs Act? ☐ Not a Panel Engineer ☐ Supervising Engineer ☐ Inspecting Engineer	
Q5.	Do you have a standard drawdown rate that you require your reservoirs to achieve during precautionary or emergency drawdown, or that you apply to the reservoirs you inspect or supervise? ☐ No standard (go to Q7) ☐ Initial drawdown rate of ………… m/day (state value) ☐ Reduction to 50% capacity in ………… days (state number of days) ☐ Hinks' formula (Low Level Outlets 1 - Formula for Target Capacity' Dams and Reservoirs Journal, 2009, 19(1), pp7-10,) ☐ Other (please specify): …………………………………………………………………… …………………………………………………………………………………………… …………………………………………………………………………………………… If your standard does not align with any of these, copies of your policy on drawdown capacity would be gratefully received. Please email/post to the address at the end of this questionnaire. All information received will be dealt with in strict confidence.	

Q6.	Does the standard in question 5 allow for reservoir inflow? ☐ Yes State criterion for inflow ☐ No …………………………………………………………… Please state any reasoning behind the inflow, or lack of inflow, used: …………………………………………………………… …………………………………………………………… …………………………………………………………… ……………………………………………………………
Q7.	In your opinion are there some reservoirs that do not require a permanent emergency outlet? ☐ Yes ☐ No If your answer is yes please indicate your reasoning or examples, which support this opinion: …………………………………………………………… …………………………………………………………… …………………………………………………………… ……………………………………………………………
Q8.	To achieve emergency drawdown do you rely on temporary pumps? ☐ Yes: ☐ No Number and size of pumps: ……………… Proportion of fixed capacity: ………………
Q9.	How many times in the last 10 years have you drawn down any of your reservoirs for precautionary or emergency purposes? ……………………………………… ☐ Emergency Drawdown Number off drawdowns: ………………. ☐ Precautionary drawdown Number off drawdowns: ………………. If you have carried out any precautionary or emergency drawdowns, please provide details of the reservoir type, reason for draw down and any important lessons learned. …………………………………………………………… …………………………………………………………… …………………………………………………………… …………………………………………………………… …………………………………………………………… ……………………………………………………………
Q10.	How many times in the last 10 years have you had a scour valve fail to operate satisfactorily (refer to guidance sheet): ………………………………………
Q11.	State any important lessons learned from drawdown exercises or events: …………………………………………………………… …………………………………………………………… …………………………………………………………… …………………………………………………………… …………………………………………………………… ……………………………………………………………
Q12.	Would guidance on defining an appropriate discharge capacity for precautionary or emergency drawdown purposes be helpful in managing the safety of your reservoirs? ☐ Yes ☐ No

<table>
<tr><td rowspan="5">Q13.</td><td colspan="4">Please rate the importance of the following factors in determining an appropriate discharge capacity for emergency drawdown purposes. using the scale of 1 to 5 below:
1 = Unimportant 2 = Not very important
3 = Fairly important 4 = Important
5 = Very important</td></tr>
<tr><td></td><td>Time to failure of the dam from first identifying a problem</td><td></td><td>How often a reservoir is visited</td></tr>
<tr><td></td><td>Concurrent inflows during drawdown</td><td></td><td>Activation time (time from being informed of a problem to starting reservoir drawdown)</td></tr>
<tr><td></td><td>Potential damage downstream</td><td></td><td>Other (please state): ....
....</td></tr>
<tr><td colspan="4"></td></tr>
<tr><td rowspan="5">14.</td><td colspan="4">We are considering three possible formats for the planned guidance, or a tiered combination of them. Please rate the following formats using the scale of 1 to 5:
1 = Not beneficial 2 = Not very beneficial
3 = Fairly beneficial 4 = beneficial
5 = Very beneficial</td></tr>
<tr><td></td><td colspan="3">A qualitative flow chart process which describes the factors that should be considered in determining an appropriate emergency drawdown capacity.</td></tr>
<tr><td></td><td colspan="3">Provision of a defined standard, or range of values, which will vary depending on the dam characteristics.</td></tr>
<tr><td></td><td colspan="3">Quantitative method to produce a site specific minimum recommended value of emergency drawdown capacity.</td></tr>
<tr><td></td><td colspan="3">Other (please state):
....
....</td></tr>
<tr><td>15</td><td colspan="4">Any other factors/points you deem to be important in determining an appropriate drawdown capacity:
....
....
....
....
....</td></tr>
</table>

Are you happy to be contacted if we would like to discuss any of your answers:

☐ Yes

☐ No

Thank you for your time in completing this questionnaire; the data will be treated anonymously and used to improve reservoir safety through the production of new reservoir guidance. Please send us your completed questionnaire by either: Email to tom.dutton@jacobs.com , or Post to: Thomas Dutton, 1180 Eskdale Road, Winnersh, Wokingham, Berkshire, RG41 5TU. We would be grateful to receive returns by 18th January 2015.

If you have any queries please contact us via email tom.dutton@jacobs.com or telephone 0118 946 8642.

Questionnaire to International Organisations

In the UK there is currently no single accepted approach to determining what represents an acceptable reservoir drawdown capacity for emergencies. A project is therefore underway to produce industry guidance on the drawdown capacity required for reservoir safety. As part of this project we are interested to learn about any standards or guidance that is used in other countries. As a representative of the international reservoir community we would be extremely grateful if you would complete the following questionnaire.

If there is insufficient space for your answer please continue on an additional sheet. If possible we would be very grateful to receive copies of any guidance/policy documents which support your answers. These can be emailed to the address shown at the end of this questionnaire. All information received will be dealt with in strict confidence.

Name:	…………………………………………………………………………………………….
Country/Organisation:	…………………………………………………………………………………………….

Q1.	In your country/organisation are there any requirements for reservoirs to have an emergency/scour outlet? ☐ No requirements ☐ No legal requirements but considered good practice ☐ Required by law/governing bodies Please provide information on any requirements/good practice: ……… ……… ……… ………
Q2.	In your country/organisation do you have a standard emergency or precautionary drawdown rate required to be met by reservoirs ? ☐ No standard (go to Q5) ☐ Initial drawdown rate of ………… m/day (state value) ☐ Reduction to 50% capacity in ………… days (state number of days) ☐ Other (please specify): …………………………………………………………………………………………… ……… ……… If your standard does not align with any of these, copies of your policy on drawdown capacity would be gratefully received. Please email/post to the address at the end of this questionnaire. All information received will be dealt with in strict confidence.

Q3. Does the standard referred to in question 2 allow for concurrent inflow into the reservoir during drawdown?

☐ Yes State criterion for inflow ………………………………………………… ☐ No

Please state any reasoning behind the inflow, or lack of inflow, used:

……

……

……

……

……

Q4. For the standard referred to in Question 2, please rate the importance given to the following factors in determining the appropriate discharge capacity for emergency drawdown purposes. using the scale of 1 to 5 below:

1 = Unimportant
2 = Not very important
3 = Fairly important
4 = Important
5 = Very important

☐	Time to failure of the dam from first identifying a problem	☐	How often a reservoir is visited
☐	Concurrent inflows during drawdown	☐	Activation time (time from being informed of a problem to starting reservoir drawdown)
☐	Potential damage downstream		
☐	Other (please state): ………………………………………………		

………………………………………………………………………………………………………

………………………………………………………………………………………………………

Q5. To achieve emergency drawdown can reservoir owners rely on temporary pumps?

☐ Yes: ☐ No

Please provide details of any restrictions on this:

……

……

……

……

Q6. State any important lessons learned in relation to drawdown capacity from drawdown exercises or events:

……

……

……

……

……

……

……

……

Q7.	Are there any other factors/points you believe are to be important in determining an appropriate drawdown capacity: …… …… …… …… …… …… ……

Are you happy to be contacted if we would like to discuss any of your answers:

☐ Yes

☐ No

Thank you for your time in completing this questionnaire; the data will be treated anonymously and used to improve reservoir safety through the production of new reservoir guidance. Please send us your completed questionnaire by either: Email to tom.dutton@jacobs.com , or Post to: Thomas Dutton, 1180 Eskdale Road, Winnersh, Wokingham, Berkshire, RG41 5TU, UK.

If you have any queries please contact us via email tom.dutton@jacobs.com

Industry survey guidance sheet

Please use the information below to assist in filling out the Industry Questionnaire. If you require further assistance please email tom.dutton@jacobs.com or telephone him on 0118 946 8642.

Key definitions

Emergency or scour outlet	A dedicated (i.e. not a water supply outlet) outlet that can discharge stored water from the base (or significantly below the top water level) of the reservoir in the event of an emergency. In some reservoirs this outlet can cross connect into the supply system and this then still falls within the scope of this definition.
Drawdown	The lowering of a reservoir's water level for purposed other than supply.
Drawdown capacity	This is the outlet capacity available to drawdown a reservoir. This is normally considered to be separate from the supply rate.
Drawdown rate	The speed at which a reservoir can be lowered, this is normally expressed as meters per day or a percentage capacity reduction in a stated period.
Precautionary drawdown	This is defined as the lowering of the reservoir because of a suspected problem that could lead to catastrophic failure.
Emergency drawdown	This is defined as the lowering of the reservoir because there is evidence of damage to the reservoir (e.g. cracking of the embankment) that could lead to catastrophic failure.

Supplementary guidance for completing the questionnaire

Question	Additional information
1	Please tick the appropriate box, if yes please state the number of reservoirs. If you own a reservoir or you are part of a club (not an employee) that owns a reservoir please check the 'owned personally' box and state the number of reservoirs you or your club operates. If the reservoir is part of the assets of the organisation which employs you please check the 'through my organisation' box, please state the number of reservoirs your organisation operates. If you Supervise or Inspect reservoirs and you are not part of any reservoir owners' organisation please check the 'No' box.
2	This information will help us to assess how many reservoirs have permanent outlets, as well as how many of these are functional. A permanent outlet will include any outlet that forms a fixed part of the reservoir and should exclude temporary measures such as temporary pumps or siphons. A functioning outlet is one you believe to be currently working and has been operated within the last 10 years. A non-functioning outlet will include those you do not believe to be working or those which have not been operated within the last 10 years.

3	If you own multiple reservoirs which are visited at different frequencies please check as many of the appropriate frequency options as required and state the number of reservoirs that relate to each option.
5	There are a range of different standards applied to determining drawdown capacity. Some of the more common ones are listed but if you use a different standard please state this in the 'other' option. If you use a reduction in capacity that is not 50%, please also use the other option and specify the reduction amount. From previous enquires we have received detailed responses to this question from the following list of reservoir owners, so if you are responding on the behalf of any of these organisations you may skip this question: Thames Water, United Utilities, Canal & River Trust, Welsh Water, Severn Trent Water, Scottish and Southern Energy (SSE) and Bristol Water
8	Some sites rely on temporary pumps to achieve a desired emergency drawdown rate. If this applies to you please state the number of pumps you require, or are likely to require, and the relative proportion of discharge rate these provide compared to any fixed emergency outlet. e.g. having a permanent drawdown capacity of $4m^3/s$ and requiring pumps to provide an additional $1m^3/s$ would equate to a proportion of fixed capacity of 25% $(\frac{1m^3/s}{4m^3/s} \times 100 = 25)$ If you have multiple sites requiring temporary pumps please specify the maximum number of temporary pumps required on a single site.
9	Please note in this question we are only interested in drawdown for precautionary or emergency purposes and not for routine maintenance or modifications (lessons learned from drawdown exercise or events can be expressed in question 11)
10	This can include the valve failing to open fully at any point including during emergencies or standard maintenance. Please only include up to one failure per valve (5 failures of the same valve should only be counted as 1 failure)
13	Please rank the following factors in terms of how important you believe each one is in determining an appropriate emergency drawdown capacity. • Time to failure of the dam: This will depend on the type of dam (e.g. puddled clay core, homogenous, rip rap covered shoulder, etc.) and the material (e.g. high plasticity clays, sandy clay, etc.) used in its construction. • How often a reservoir is visited: this may be a factor in the delay between a problem occurring and actions being undertaken. A problem may progress without anyone noticing. • Concurrent inflows during drawdown: This is the flow into the reservoir during the period of drawdown. The rate of drawdown will be determined by the drawdown capacity minus the inflow. The reservoir will not drawdown if the inflow is higher than the drawdown capacity. • Activation time: this is the time delay from someone spotting a potential problem with the dam to the time when drawdown is started (e.g. the opening of a valve).

	• Potential damage downstream: the appropriate drawdown capacity could be related to the potential downstream impacts (e.g. where there is a high potential for damage downstream the guidance could recommend a higher drawdown capacity compared to a reservoir with little downstream development). • Other: If you believe there is another major factor that should be considered in determining emergency drawdown capacity, please state this and rate its importance
14	Can you please rate the following 3 proposed methods based on how useful you would find each one: • **A qualitative flow chart process**: this would be a flow chart which highlights the key factors that need to be considered in determining emergency drawdown capacity. It will not state how calculations are done and will not give a numerical value on what drawdown rate to use. • **Provision of a defined standard**: this is likely to be in the form of a table where the rows and columns of the table are categorised for different situations and one value can be taken from the table. This is likely to be a conservative but simple method to calculate the required rate. • **Quantitative method**: this will be a set of simple calculations used to produce a recommended minimum drawdown rate.

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