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Guide to drawdown capacity for
reservoir safety and emergency
planning

SC130001 Volume 1 – main guide

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Professor Doug Wilson
Director, Research, Analysis and Evaluation

Executive summary

In England, 1.1 million properties are at risk of flooding from the structural failure of large raised reservoirs and their associated dams. The average age of these structures is 120 years and the possibility of a catastrophic failure may be expected to increase with age. A key factor in avoiding and minimising the impact of such a catastrophic failure is the ability to draw a reservoir down in the event of an emergency. This will reduce the load on the dam structure, reduce the likelihood of failure and, in the very worst outcome, minimise the impacts downstream in the event of failure.

This publication provides guidance on:

- types of drawdown facility and general considerations for designing, maintaining and operating them (Section 2)
- characterising a reservoir site in order to evaluate the drawdown capacity (Section 3)
- determining the existing drawdown capacity, taking into account concurrent inflows and the reliability (Sections 4 and 5)
- determining an appropriate drawdown capacity for reservoirs in the UK (Sections 6 and 7)
- mitigation measures where existing facilities do not meet this capacity (Section 8)

The drawdown capacity is made up of two components, the reservoir lowering capacity and the inflow pass-through allowance. The general standard recommended for the inflow pass-through allowance is the Q_{50} (i.e. the daily inflow to the reservoir that is exceeded on average 50% of days in a year) but sensitivity checks are recommended to consider how higher inflows such as the Q_{10} could affect the ability to lower the reservoir.

The approach for assessing whether the installed drawdown rate is adequate should be based on judgement by an experienced dam engineer taking into account various considerations. For embankment dams (Section 6), basic minimum recommended standards are proposed for the rate of drawdown, which vary depending on the potential consequences of the dam failing. These standards are based on a number of assumptions which should be reviewed as part of the assessment. For example the standards may need to be adjusted depending on the vulnerability of a dam to rapid failure, and the time it may take to detect symptoms of failure and to activate drawdown. The assessment should also consider the time it would take to lower a significant proportion of the reservoir depth (normally one-third) and the ability to keep the reservoir drawn down to enable repairs. Precedent practices may also be taken into consideration as part of the judgement.

For concrete and masonry gravity dams, and service reservoirs, the potential failure modes are different and specific guidance is provided to reflect this (Section 7).

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1 Introduction

1.1 Purpose of guide

In England, 1.1 million properties are at risk of flooding from the structural failure of large raised reservoirs and their associated dams. The average age of these structures is 120 years and the possibility of a catastrophic failure may be expected to increase with age. A key factor in avoiding and minimising the impact of such a catastrophic failure is the ability to draw a reservoir down in the event of an emergency. This will reduce the load on the dam structure, reduce the likelihood of failure and, in the very worst outcome, minimise the impacts downstream in the event of failure. Reservoir drawdown is also important to allow inspection and maintenance of the structures retaining the reservoir. It should be noted, however, that while inclusion of drawdown facilities in accordance with this guidance document is recommended as good practice, it cannot be expected to be successful in averting an imminent failure under all circumstances.

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.

This document provides guidance on a consistent methodology for assessing the adequacy of existing drawdown capacity at reservoirs in the UK. It outlines the considerations which should be taken into account as part of an overall judgement to evaluate this aspect of reservoir safety. The guidance in this document is not statutory; however, it is recommended that where an engineer feels it is right to depart from the general principles within this guide, the reasons for the departure should be presented in the assessment.

The guide is principally aimed at assessing drawdown capacity at existing reservoirs because these types of assessments form the majority of work in this field. However, the guide may also be used for the design of drawdown facilities at new reservoirs and as a reference in relation to emergency planning.

Because embankment dams are the predominant dam type in the UK and have a common set of failure modes they are the prime focus of the guide, but Section 7 gives guidance on other common dam types that may have different failure modes – specifically concrete and masonry dams and service reservoirs.

In addition to determining an adequate rate of drawdown, the guide also discusses issues such as:

- different types of drawdown facilities, their application and risks of operation
- options for mitigating the risks where the existing drawdown capacity is judged to be insufficient

This volume of the document is the main guide. It contains all of the key information and guidance which is likely to be needed for the majority of assessments. Volume 2 of this guide is published as a separate document and provides background and supplementary information related to the derivation of the guidance presented in this volume.

1.2 Function of drawdown facilities

Drawdown facilities can provide a means to lower a reservoir's level quickly in an emergency in the event that a structural problem occurs which threatens, or potentially threatens, the safety of the dam. This may be a precautionary measure while the problem is investigated, or an emergency measure. In either case, the primary objective would be to reduce the load on the dam, and thereby arrest a failure mode which has already initiated, or is at high risk of initiating, and prevent it from developing. If this objective cannot be achieved then partial drawdown may at least buy time to make repairs, or evacuate downstream, or employ other techniques to avert failure. In the very worst outcome the intervention of drawdown may at least reduce the consequences of failure by reducing the volume of water released in a breach.

In the period after an emergency drawdown, the drawdown facilities may allow the reservoir level to be controlled while repairs are carried out.

The operation of drawdown facilities will contribute to inhibiting the initiation and progression of several potential failure modes (see Section 3.5), although it should be noted that their purpose is not to mitigate against failure from flood discharges; this is the function of the spillway.

The common failure modes for concrete and masonry dams differ in a number of ways from embankment dams (see Section 7.2.3), and as a result the assessment of the appropriate capacity of drawdown facilities for these dam types will be considered separately in Section 7.

A key principle of this guide is that increasing drawdown capability should not be used as an alternative to rectifying dams in poor condition. Drawdown capacity should not be considered as justification for accepting serious deficiencies in the dam design, its condition, or maintenance or surveillance practices. Drawdown capability is a safety provision which provides an emergency fall back measure. Any known serious defects at a dam should always be rectified regardless of how good the drawdown capacity might be.

Existing facilities in dams that can be used for drawdown may have been installed for various purposes, including:

- to improve reservoir safety by ensuring water levels can be lowered quickly in an emergency, in the event that a structural problem occurs which threatens the safety of the dam
- to allow a reservoir to be drawn down for inspection and maintenance, either routinely or to investigate a specific problem
- in low-level outlets, to serve various other functions as discussed in Section 2.1.2 (e.g. they may be left over from the original dam construction as a means of temporary river diversion, or they may be designed to scour silt from the reservoir bed)

This guide is concerned with the capacity of drawdown facilities for reservoir safety purposes. However, it is acknowledged that, for many older dams, this may not have been the original design intent of the existing installation.

Although this guide focuses on the benefits of drawdown facilities in terms of inhibiting the initiation and progression of failure modes, conduits through dams are also a vulnerable feature in themselves where there is a greater risk of internal erosion developing. In certain circumstances this may be a reason for not providing a low-level outlet through the embankment as it removes one of the potential risk pathways. It should also be noted that retrofitting facilities can have significant dam safety implications.

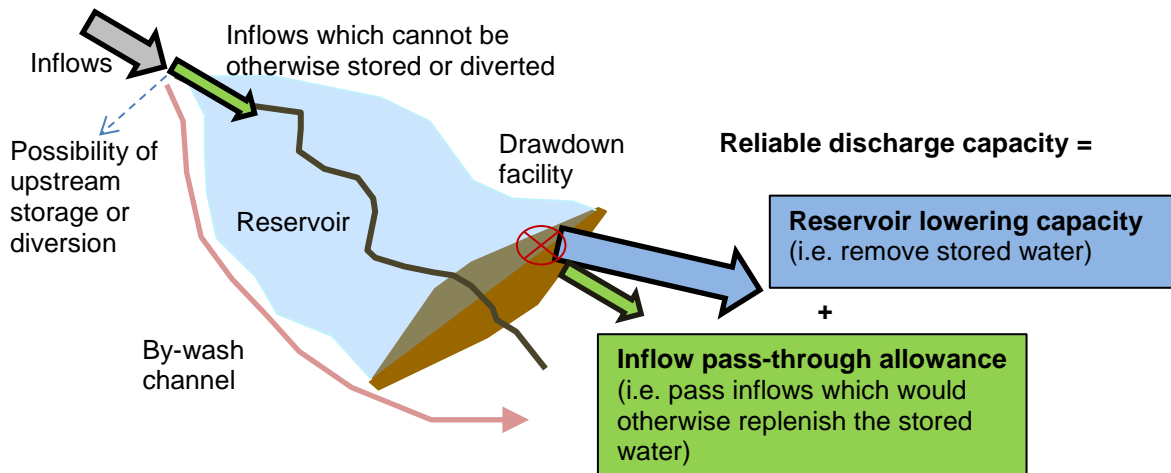
1.3 Outline of approach in this guide

There are two components which make up the total drawdown capacity for a reservoir:

- reservoir lowering capacity
- inflow pass-through allowance

Where a by-wash channel exists, or there are other means of storing or diverting some or all of the normal inflows around the reservoir, then the inflow pass-through allowance may be reduced accordingly. This concept is illustrated in Figure 1.1.

Figure 1.1 Concept of reservoir lowering capacity and inflow pass-through capacity

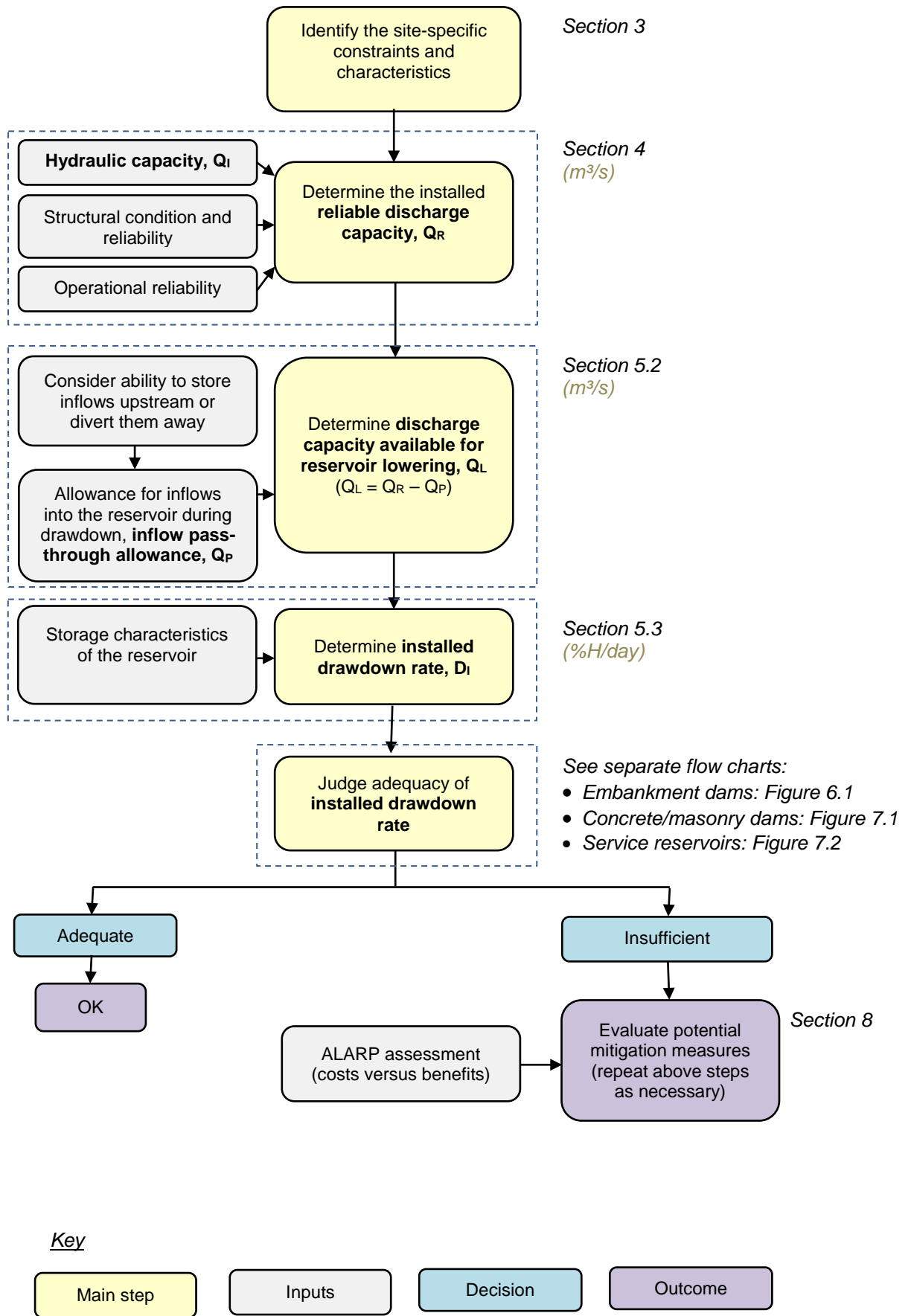


Sections 4 and 5 of this guide provide guidance on determining the existing drawdown capacity at a reservoir, after making allowance for reservoir inflows. Section 6 (or Section 7 for concrete and masonry dams) then presents an approach for assessing whether this capacity is adequate. Where the capacity is judged to be insufficient, guidance is given on developing appropriate mitigation measures in Section 8. An outline of the approach adopted in this guide is illustrated in Figure 1.2.

The following nomenclature is adopted throughout the guide:

- Q is used to denote discharge capacity (m^3/s).
- D is used to denote drawdown capacity, i.e. reservoir lowering capacity (either m/day or % of maximum reservoir depth per day).

Figure 1.2 Outline of approach in this guide



2 General considerations

2.1 Types of drawdown facility

2.1.1 Overview

This subsection describes the types of drawdown facility commonly installed at UK reservoirs. It outlines the principal characteristics of each type and gives points to consider when assessing the available capacity. A key consideration should always be whether or not the facility is accessible and reliable at short notice in an emergency. Routine testing and exercising of facilities is an important activity in this regard as described in Section 2.3.

Some reservoirs may have more than one type of drawdown facility which can be used in combination. Indeed, where possible, a degree of redundancy is beneficial in the operation of drawdown facilities in case there is a problem affecting one particular element of them. For example it is good practice to have a cross-connection between the scour and draw-off pipework.

2.1.2 Permanent dedicated facilities

The following types of facilities are permanently installed at a reservoir for the sole purpose of providing drawdown capacity. They offer significant advantages over temporary or non-dedicated facilities in respect of being immediately available in an emergency.

Low-level outlets

The most common form of drawdown facility is a low-level outlet pipe through the dam or its abutment. There are many types of 'low-level outlet', of all shapes and sizes, at different locations and with different valving arrangements. Low-level outlets normally date back to the original dam construction as their later addition can be disruptive, costly and have significant dam safety implications.

Low-level outlets are not necessarily solely dedicated to emergency drawdown and may also serve one or more of the following purposes, either as their primary or secondary function:

- To scour silt from the reservoir bed, particularly to mitigate the risk of other draw-off outlets becoming blocked. These pipes are often called scour outlets and usually discharge into the watercourse directly downstream of the reservoir. Although this may have been their original purpose, in practice they are generally ineffective at scouring silt other than locally around the intake.
- For drawing water off into supply (e.g. for water supply or hydroelectric power generation). Draw-off outlets are often provided at different levels within the reservoir, one of which is at the base of the reservoir and forms a low-level outlet. The draw-off outlets normally discharge directly into a water treatment works or other facility some distance downstream of the dam, sometimes with a branch discharging directly to the river.
- To provide compensation water to the downstream watercourse. These outlets are frequently tapped off the main supply pipework (see above) with a branch to allow discharge into the watercourse.

Service reservoirs tend to have a low-level wash-out pipe for draining the reservoir, which often connects into the overflow pipework, although the capacity of these tends to be relatively small compared to the draw-off pipework.

CIRIA Report C743 (CIRIA 2015) gives guidance on issues associated with conduits through dams including inspection, monitoring, investigation, maintenance and repair. It highlights the risks of conduits through dams creating a pathway for seepage, thus increasing the risk of internal erosion, and describes measures to mitigate this risk. CIRIA Report 170 (CIRIA 1997) gives more specific guidance on different types of valve and pipework.

When evaluating the drawdown capacity provided by low-level outlets, the following key issues should be considered:

- What is the level of the outlet relative to the depth of the reservoir? This will govern the available head and thus the outlet capacity (see Section 4), as well as the depth to which it is possible to lower the reservoir (see Section 2.7).
- Is there a risk that the upstream end of the outlet could become silted up? Regular exercising may mitigate this risk (see Section 2.3).
- Can the valves be safely and reliably operated in an emergency? For example in the event of a structural problem with the dam it may not be safe to send operatives into confined spaces within the dam. Another example may be where access to operate the valves is via a weak access bridge which may be vulnerable to damage in floods or earthquakes.
- Multiple smaller outlets can offer advantages over a single larger outlet. For example it provides redundancy in the event of a problem with one pipe or valve and the environmental impacts on downstream watercourses can be reduced by exercising them individually.
- Many low-level outlet valves (i.e. those discharging to the atmosphere, rather than into a network) are subject to higher velocity flows than those normally experienced in water mains, especially when valves are only partially open, and may be vulnerable to cavitation which can erode the pipe wall or edges of a valve gate over time. In some cases, operating valves partially open may also cause dangerous levels of vibration. Further detail on cavitation is provided in references such as Lewin (2001). It is normal practice on higher dams to provide specialist valves designed to discharge to the atmosphere, such as cone discharge valves, to prevent cavitation and vibration issues.

Siphons

These are pipes laid either up and over or through the dam to siphon water out of the reservoir. They are often buried at shallow depth through the crest to avoid disrupting crest access. The discharge point is normally lower than the inlet to provide increased discharge capacity when the pipe operates full. A typical siphon arrangement is shown in Figure 2.1. Siphons are often the least disruptive and most cost-efficient solution for retrofitting additional drawdown capacity to existing dams. Another advantage of siphons over low-level outlets is that they are more accessible for inspection and maintenance and reduce the dam safety risks associated with buried conduits through the base of a dam.

The disadvantage of siphons is that they often need to be primed using an external pump to start the water flowing (unless the pipe through the dam crest is laid just below top water level). Options for priming are summarised in Table 2.1.

Another limitation of siphons is that there is a maximum depth to which they will work of around 5m. Given that the average depth of UK reservoirs is around 12 to 15m this is often not a particular issue. When the depth between the siphon crest and the reservoir

water level reaches around 5m bubbles start to form which fill the pipe and break the siphon. The problem is exacerbated because as the liquid is raised through the siphon the pressure drops, causing dissolved gases within the liquid to come out of solution (see Appendix B.2). Generally siphons provide a lower discharge capacity than a similar sized low-level outlet.

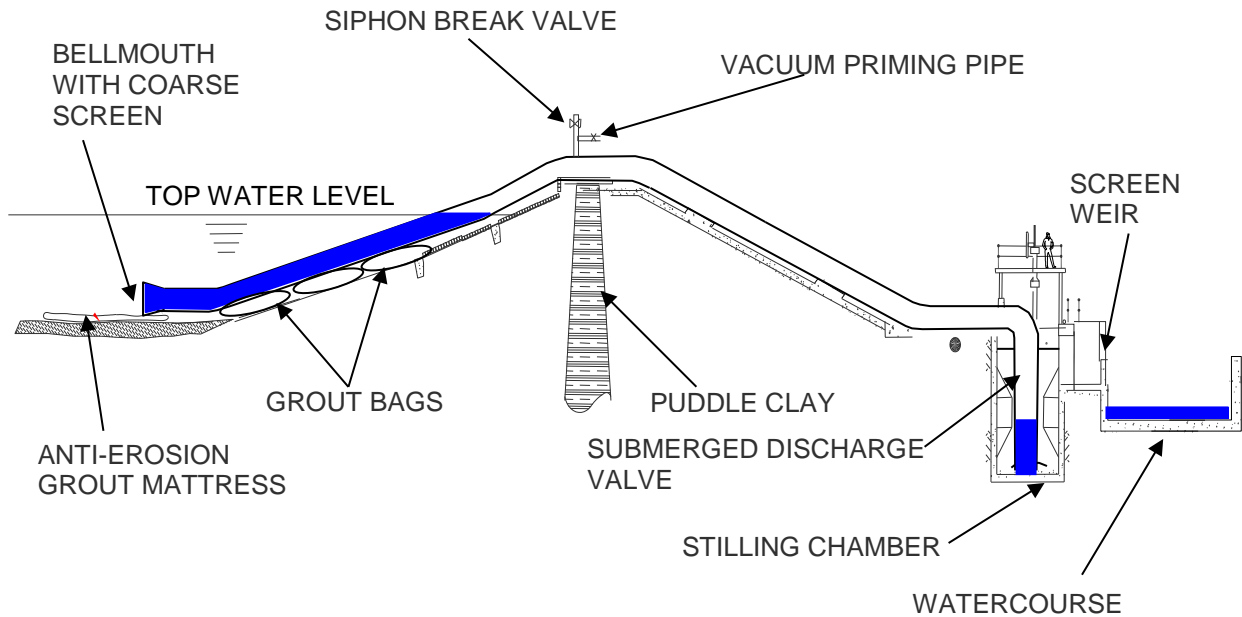
Another important consideration is that the inlet to the siphon needs to be below the design draw-off level by an amount sufficient to ensure full submergence, such that vortices cannot form and lead to air entering the pipe (and breaking the siphon). The depth of submergence required depends on the pipe diameter and velocity, and may be established from published equations. For similar reasons the pipework needs to be completely airtight with no leaks.

Technical papers by Head (1971, 1975), Ackers and Thomas (1975) and Ackers and Ashraf Akhtar (2000) provide more information, together with standard hydraulic textbooks.

Table 2.1 Options for priming a siphon

Option		Description	Comments
1	Locate pipe below top water level (TWL)	Pipe laid in crest, with crown just below TWL, so it can be filled by gravity when the reservoir is full.	Possible increased risk of leakage along outside of pipes. A disadvantage of this is that the siphon can only be operated when the reservoir is full unless an alternative method of priming is also installed.
2	Water priming	Valves on the upstream and downstream end of the siphon are initially closed and the siphon pipework is filled with water using a pump. The upstream valve and downstream valve are then opened (in that sequence) to initiate the siphon.	Robust and reliable method. Requires valves on both ends of the pipe and a means of pumping water into the pipework. On large-diameter siphons it may take some time to activate the drawdown.
3	Vacuum priming	The valve on the downstream end of the siphon is initially closed and air is pumped out of the pipework to create a vacuum and ultimately draw water into the pipework from the reservoir. The downstream valve is then opened to activate the reservoir drawdown.	This avoids the need for an upstream (submerged) valve. This method of priming can be slower, particularly for large-diameter siphons, and is susceptible to non-sealing of the siphon system. Pipe walls may need to be thicker to resist the vacuum pressure.
4	Combination of the above two methods (i.e. two-stage process)	Using a valve in the crest, the pipework downstream of this is water primed and the pipework on the upstream face is vacuum primed.	This optimises the advantages and disadvantages of both methods. Water priming can be undertaken at the first stage of a suspected emergency. For example this is the system used at Queen Mary Reservoir (Philpott et al. 2008).

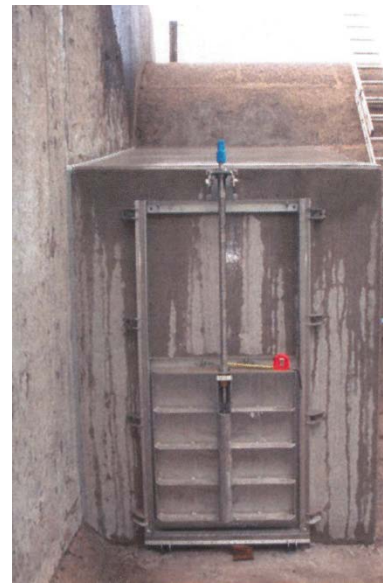
Figure 2.1 Typical siphon arrangement as adopted at Queen Mary and King George V reservoirs (courtesy of Thames Water)



Gates in spillway weirs

A convenient location to install outlets, particularly if they are being retrofitted to an existing dam, is through the spillway weir wall, with a penstock gate discharging into the tumblebay at the top of the spillway chute (see Figure 2.2). In these cases, the elevation of the outlet is generally relatively high compared with the depth of the reservoir, so the available head and thus drawdown capacity is often limited. However, gates in spillway weirs remain a useful and cost-effective means of drawing down the upper levels of the reservoir. Outlets can also be installed through the shaft wall of bellmouth spillways.

Figure 2.2 Outlet being retrofitted through spillway weir wall (Northern Ireland Water, 2014(b))



Stop logs

At lower height dams, drawdown capacity may be provided using stop logs built into the dam structure (Figure 2.3). In such cases the stop logs may also act as the spillway. A particular challenge with stop logs is ensuring that they can be removed safely in an emergency, especially if they have not been removed for some time and where access is difficult. Lifting equipment (e.g. an excavator) may be required to lift them out, in which case consideration should be given to lifting points on each plank and adequate access for the excavator to get to the stop logs.

Figure 2.3 Stop logs used for drawdown capability at a low dam



2.1.3 Draw-off of water for operational use

The largest-sized outlet from a reservoir is often the operational draw-off outlet used to provide water into supply (e.g. to a water treatment works or hydroelectric power station). This is especially the case for most service reservoirs. For water supply reservoirs these are often multi-level draw-offs, allowing the level from which water is drawn off to be varied to suit water quality, which normally varies with depth in the reservoir. However, the capacity of such draw-off outlets may, at times, be restricted by the throughput capacity of the works downstream. Therefore, when assessing the existing drawdown capacity, discharge through operational draw-offs should only be taken into account if it is reliably available in an emergency situation. In particular, before including operational draw-off, the following considerations need to be taken into account:

- What is the normal throughput of the downstream water treatment works or power generation plant (i.e. what rate of discharge can be reliably passed through the works on say 90% of days in the year)?
- How long would it take to ramp the process or generation plant up to its maximum throughput in an emergency? If this can be reliably achieved within a reasonable activation time then it may be acceptable to take into account the full plant capacity with an appropriate factor applied for the activation time (see Section 6.5.2).
- What is the procedure for draw-off into supply? Would output from other reservoirs or sources potentially need to be reduced to balance the flow through the works? Are the appropriate personnel available at all times to achieve this? Can all this be undertaken safely in an emergency situation at

short notice? The procedures and details should be planned and documented in an on-site emergency plan.

- Is there a risk that the reservoir defect for which drawdown is required may adversely impact the quality of the water within the reservoir such that it would cause significant problems for the operational network or plant downstream? For example, if part of a service reservoir roof collapsed and earth entered the reservoir, it may be unacceptable to release the reservoir water into the public drinking supply on water quality grounds. At the very least such issues could cause delays in decision-making and in some cases they may prevent draw-off for operational use.

The pipework between the reservoir draw-off and the water treatment works, power station or canal may include a branch or wash-out valve (separate to any wash-out provided for the reservoir itself). Opening such valves may provide a valuable addition to the available drawdown capacity. New wash-out valves of a larger diameter can be retrofitted to supply mains as an effective solution to increasing drawdown capacity.

In many cases water treatment works or power stations, and the associated operational pipework and valves, are located immediately downstream of the dam and may therefore need to be evacuated in an emergency drawdown scenario if there is a risk of the dam failing. If this is the case and the necessary valves would be inaccessible then it would not be appropriate to rely on this as operational drawdown capacity.

2.1.4 Temporary and emergency measures

In some cases, temporary drawdown facilities and/or emergency methods may be used as a supplement to, or as an alternative to, the permanent installations described above. Options for temporary or emergency drawdown facilities are outlined in Table 2.2.

Table 2.2 Options for temporary and emergency drawdown measures

Option	Description/comments											
Mobile pumps	<p>This is the most commonly relied on temporary option for reservoir emergency planning. Some undertakers have their own pumps, or otherwise they can be hired from various hire companies, delivered on a Hiab lorry, and set up on the dam abutment or crest. The time required to mobilise and set up the pumps will depend on the number and size of pumps required, the accessibility of the site and the quality of emergency planning in place. Adequate access for lorries and cranes needs to be available if mobile pumps are to be relied on. Generally pumps have a relatively small capacity compared with permanent dedicated facilities or drawdown into supply. For example typical pump capacities are:</p> <table border="1" data-bbox="472 562 1399 792"> <thead> <tr> <th data-bbox="472 562 740 651">Pump size</th> <th data-bbox="740 562 986 651">Maximum flow rate (l/s)</th> <th data-bbox="986 562 1399 651">Typical mobilisation time in an emergency from hire depot (excludes journey time)</th> </tr> </thead> <tbody> <tr> <td data-bbox="472 651 740 696">150mm (6 inches)</td> <td data-bbox="740 651 986 696">55</td> <td data-bbox="986 651 1399 725" rowspan="2">Around 4 hours</td> </tr> <tr> <td data-bbox="472 696 740 725">200mm (8 inches)</td> <td data-bbox="740 696 986 725">95</td> </tr> <tr> <td data-bbox="472 725 740 792">250mm (10 inches)</td> <td data-bbox="740 725 986 792">250</td> <td data-bbox="986 725 1399 792">Not commonly in stock, therefore around 2 days</td> </tr> </tbody> </table> <p>The above rates were validated in an exercise at Pebley Reservoir (Windsor 2012) and the paper suggests that the capacities are similar for both submersible pumps and modern suction pumps, up to heads of around 6m. The rates from an 8-inch electric submersible pump were also validated by the Canal & River Trust using a thin-plate weir during an exercise at Foulridge Reservoir (Brown et al. 2010). The above rates are significantly lower than the capacities quoted by pump manufacturers and the latter should therefore be treated with caution.</p> <p>While an emergency plan may express a preference for a particular type of pump (e.g. submersible or suction), the reality is that in a real emergency it will be a case of using whichever is more readily available and can be quickly and safely deployed. In the initial stages this is likely to be suction pumps.</p> <p>In response to widespread flooding in Somerset and the north of England in 2013/14, the Environment Agency acquired 20 ultra-high volume pumps. This includes 3 BBA pumps, with a capacity of 1650, 1500 and 1100l/s and 17 1000l/s pumps, 10 of which are submersible. They can be operated by the Environment Agency and its local resilience forum partners and can be made available to other emergency services during a flood or reservoir emergency. For example, during the Cumbria and Lancashire floods in December 2015, a 24-inch 1000l/s pump and all associated pipework was deployed in Carlisle.</p>	Pump size	Maximum flow rate (l/s)	Typical mobilisation time in an emergency from hire depot (excludes journey time)	150mm (6 inches)	55	Around 4 hours	200mm (8 inches)	95	250mm (10 inches)	250	Not commonly in stock, therefore around 2 days
Pump size	Maximum flow rate (l/s)	Typical mobilisation time in an emergency from hire depot (excludes journey time)										
150mm (6 inches)	55	Around 4 hours										
200mm (8 inches)	95											
250mm (10 inches)	250	Not commonly in stock, therefore around 2 days										
Mobile siphons	<p>This technique is relatively rare but can offer several advantages over high-capacity pumps. The approach is based on the same principles described in Section 2.1.2 but, instead of permanently installed pipework, multiple siphons are formed using pump hoses (typically 150mm (6 inch) or 200mm (8 inch) diameter hired from a pump supplier) or rigid pipes laid into the reservoir and over the dam crest. Advance planning is required to determine the best way of installing and priming the siphons.</p>											
Emergency breach	<p>This approach is to excavate a notch at a suitable point in the dam, normally a low point near one of the abutments, in order to lower the water level. This approach is only acceptable where a suitable location for the notch can be identified where there is no risk of it eroding and propagating into a full-scale dam breach. Emergency planning is therefore crucial and should be carried out in consultation with a qualified reservoir engineer, such as the reservoir panel engineer for reservoirs registered under UK reservoir legislation. It is normally best suited for relatively small dams but may also be appropriate for some larger dams.</p> <p>An example of an emergency breach being implemented was at Combs</p>											

Option	Description/comments
	Reservoir in 1976 (CIRIA 2014).

None of the options shown in Table 2.2 are likely to be totally effective in an emergency unless they have been properly planned as part of an on-site emergency plan (see Section 2.2). Among other things, the emergency plan should consider:

- what pumps or other equipment may be required, including capacities and lengths of hosing
- where such equipment would be sourced, including during out-of-hours periods (note larger hire companies often offer 24-hour call-out services but these may not be from the local depot)
- how the site would be accessed
- details of how any pumps etc. should be set up including where the hoses should safely discharge and how the pumps would be maintained and refuelled
- the level of site manning and security

Reservoir sites are often located in remote areas, with narrow single track access, which can make deliveries of (potentially multiple) large pumps and equipment in an emergency difficult.

Where significant reliance is placed on pumps or mobile siphons it is good practice to carry out a test exercise. Such an exercise was carried out by the Canal & River Trust in November 2011 at Pebley Reservoir in Derbyshire, and this identified a number of practical lessons (Windsor 2012). For example, it was concluded that ten 8-inch pumps (providing approximately 1m³/s total capacity) is the maximum practical number of temporary pumps for a typical site.

Because they require significant advance planning, take longer to activate and generally provide a much smaller drawdown capacity than permanent facilities, temporary and emergency options are generally best used as a supplement to fixed drawdown facilities. However, Section 6.3.2 identifies situations when, in terms of the risk to public safety, it may be appropriate to have no permanent drawdown facilities and to rely solely on temporary and emergency facilities. Even though permanent facilities may not be required in these cases for public safety, they may still be desirable in terms of preserving the reservoir asset and water supply, and protecting against the reputational loss associated with a dam breach.

2.2 Emergency planning

Emergency plans can be effective in preventing failure in the event of emergencies affecting the structure of the dam, or if failure cannot be prevented and a warning can be given, in reducing the number of injuries and fatalities. An example of such an emergency is leakage from the dam which accelerates due to associated erosion of fill from within the body of the dam.

There are two types of emergency plans for reservoirs:

- **On-site (flood) plans**, which are the responsibility of the reservoir owner, and set out measures to be taken on site in an emergency to try to prevent the dam from failing, or if this is not possible, to contain and reduce the consequences of failure.

- **Off-site plans**, which are developed by local authorities to ensure communities are well prepared. In particular, they set out what the emergency services will do to warn and protect people and property if a dam or reservoir were to fail. It is considered good practice to have established contacts with the emergency planning officer in the local council as part of reservoir safety management, and to have established which (if any) dams have site-specific off-site plans and which would have to rely on a generic plan prepared by that local authority.

National guidance for producing an on-site plan, including downloadable report templates, is available on the following website: <https://www.gov.uk/guidance/flood-risk-management-information-for-flood-risk-management-authorities-asset-owners-and-local-authorities>

Further background information is given in the informal draft consultation document on emergency planning for reservoirs (Defra 2006) and the paper by Morris and Shakespeare (2015).

The method of rapidly lowering the reservoir at short notice should be a key aspect of any on-site plan. The plan should set out the procedure for drawdown assuming the reader has no prior knowledge of the reservoir. A drawing should be included to show where key aspects of the dam (e.g. emergency valves) are located and how they are accessed. Where the plan involves deploying temporary or emergency measures the issues raised in Section 2.1.4 should be planned and documented.

The Water Act 2003 gives the Secretary of State power to direct owners of reservoirs to produce on-site plans, so it is possible that in the future it will be legally mandatory for reservoir owners to produce such plans.

2.3 Routine exercising of facilities

If not operated routinely there is a risk that valves and penstocks may become seized and therefore not be available in an emergency. Similarly, low-level outlets may become silted up with equal consequences. To mitigate these risks it is recommended that valves and gates on all drawdown facilities are regularly exercised. Many reservoir owners exercise the valves at 6-monthly intervals. For reservoirs registered under UK reservoir legislation, regular valve exercising is frequently a statutory direction by the inspecting engineer. It is good practice to maintain a record of valve operations along with a comment on the ease of operation and any issues found. For reservoirs designated as 'high-risk' under the Reservoirs Act 1975 it is a legal requirement to record such details in Part 16 of the Prescribed Form of Record.

Valves and gates should preferably be exercised over their full range of travel (i.e. fully opened and closed again). If valves are only exercised over part of their range there is a risk they will not open beyond this range in an emergency. Other hazards associated with operating valves partially open are discussed in Section 2.1.2 (Low-level outlets).

When exercising valves they should ideally be left open for a few minutes, until the discharge becomes clear, in order to flush any silt or debris through the outlet and thus ensure a good seal when the valve is closed again. However, it is acknowledged that in some cases fully opening a low-level outlet may cause localised flooding downstream or cause undesirable environmental impacts. These issues need to be managed as described in Sections 2.4 and 2.5 and should not generally be an excuse for not properly exercising drawdown facilities. However, this may be easier said than done in some situations, and where major issues prevent routine exercising of critical valves under full head then advice should be sought from an inspecting engineer.

Where low-level outlets are fitted with multiple valves (e.g. an upstream guard valve and a downstream control valve) it is possible to exercise each valve independently without actually releasing any significant discharge and this can be useful to mitigate downstream impacts during routine exercising. However, there is a risk that this approach can cause the pipe to become filled with silt and it is recommended that outlet valves should be exercised under full head conditions (i.e. with full discharge being released) on a sufficient number of occasions to establish and maintain confidence that the discharge can be achieved. This is because while valves may operate satisfactorily under balanced head conditions there are more likely to be issues operating them under full head conditions.

2.4 Environmental permitting

Operating low-level outlets can have an adverse impact on downstream watercourses for a number of reasons and therefore releases from reservoirs need to be managed under environmental legislation. The primary legislation in England and Wales is the Water Industry Act 1991, Sections 165 and 166, with the relevant secondary legislation being the Environmental Permitting (England and Wales) Regulations 2010 (EPR 2010). Similar principles apply in Scotland under local legislation.

An explanation of the procedure, steps to take, environmental risks and legal remit can be found within Annex 9, pages 199 to 214 of the Environment Agency guidance *How to comply with your environmental permit. Additional guidance for: Water Discharge and Groundwater (from point source) Activity Permits (EPR 7.01)*, which is available on the following website: <http://www.gov.uk/government/publications/water-discharge-and-groundwater-activity-permits-additional-guidance>.

Annex 9 of the guidance is currently being reviewed to provide greater clarity and comply with Gov.uk publishing rules, and the revised version is expected to be published on the above website soon.

EPR 2010 applies to all reservoir undertakers in England and Wales although different reservoir undertakers are subject to different legislative provisions. All reservoir undertakers need to apply for a consent or permit for any planned drawdown operation through the scour valve and any other releases that may discharge solids or other pollutants. Only water undertakers (i.e. companies licensed by the regulator to supply drinking water) can apply for discharge consents under Section 166 of the Water Industry Act 1991. All other undertakers would need to apply under EPR 2010 if their activity meets the definition of water discharge activity.¹ The undertaker's risk assessment and proposed mitigation measures plan will form the main part of the application.

If a dam is in serious danger of failing, the priority must be to lower water levels in the reservoir as quickly as possible in order to prevent an uncontrolled release of water, which could cause widespread flooding and potentially loss of life. In such emergency circumstances environmental matters should not be a reason for unduly delaying the drawdown. Regulation 40 of EPR 2010 does, however, require that in such emergency circumstances all reasonably practicable steps should still be taken to minimise pollution, and that the particulars of the discharge should be provided to the regulator as soon as reasonably practicable afterwards.

The key considerations which need to be considered in order to comply with an environmental permit are as follows:

- Minimising the release of sediment.

¹ The definition for 'water discharge activity' in EPR 2010 includes 'the removal from any part of the bottom, channel or bed of any inland freshwaters of a deposit accumulated because of any dam by causing it to be carried away in suspension in the waters'.

- Preventing the release of any polluted sediment.
- Managing water quality issues. For example:
 - Thermal stratification can mean that water at the bottom of a reservoir may have low dissolved oxygen which if discharged can cause pollution, killing fish and other animals. This is one of the most environmentally damaging aspects of reservoir releases. A dissolved oxygen probe should be lowered through the water to check that there is at least 50% of the air saturation value in the deep water before making a release.
 - Large amounts of algae (algal bloom) in the reservoir may also cause poor quality discharge water. Planned discharges should not be made if there is a severe algal bloom, unless the water being released is deeper and unaffected.
- Fish may be washed from the reservoir with the discharged water. Some reservoir owners crack the valve initially and leave it for a while before continuing to encourage fish to move away from the outlet. The Environment Agency does not normally require any significant mitigation measures for fish displacement during scour valve tests but it may do for substantial reservoir drawdowns.
- Trying to time any releases as best as possible with natural flow patterns. Extreme sudden high flows can cause channel and bank scouring and flooding and should be avoided. Scour valve releases that simulate as closely as possible the natural flow patterns of the river are the best for maintaining as natural an environment as possible.
- Managing the increase in downstream flow (see Section 2.5).

2.5 Risk of localised downstream flooding

At some reservoirs, operation of the drawdown facilities can cause localised flooding downstream of the reservoir. While this could potentially damage property, for UK reservoirs it would rarely endanger life and therefore in a genuine emergency it is normally better to fully operate the facilities and suffer some localised flood damage, than risk the far greater consequences of dam failure which may occur if the drawdown rate is restricted.

The risk of localised downstream flooding is, however, much more of an issue in terms of routine exercising of drawdown facilities. Section 2.3 explains the importance of routinely testing drawdown facilities, so the risk of downstream flooding needs to be managed to allow routine testing to take place.

The first step in managing this risk is to understand the problem (i.e. which properties are affected, to what extent and under what conditions). For example, flooding may only occur if the outlet is operated during high natural river flows, or if it is operated for a prolonged period. In some cases flooding is cited as a reason for not fully operating outlets without really knowing the details or even if it is a genuine problem. Once the details of the problem are quantified either by a hydraulic study or through controlled trials, it may be possible to develop a strategy for exercising drawdown facilities which mitigates the risk. Possible strategies may be to control other discharges further upstream during the tests, or only test facilities during periods of low flow in the river, although this latter approach would contravene normal good environmental practice (see Section 2.4) and should be discussed with the environmental regulator as part of the discharge permit.

If an acceptable testing strategy cannot be developed to mitigate the risk of flooding, then other physical measures should be taken to ensure valves can be routinely exercised over their full range. A possible measure might be to install two valves in line on the outlet so that each can be fully exercised in turn without making any significant releases. This technique is not ideal as it does not test the valves under load and there is a risk silt may be drawn into the outlet, so ideally outlets should be tested by releasing water until the discharge becomes clear. However, in some cases operating two valves independently in sequence may be the only practical option and may be preferable to partially operating valves for the reasons discussed in Section 2.1.2.

2.6 Effects of rapid drawdown on slope stability

At some embankment dams rapidly lowering the water level can cause instability of the upstream dam slope or reservoir rim, due to pore water pressures remaining high in the soil, after the supporting reservoir water is removed. This issue depends very much on the permeability of the soil in the upstream shoulder and/or reservoir banks. The problem is greatest in clay fills where pore pressures in the soil take a long time to dissipate. With rockfill or other free-draining materials, and fairly open face protection, pore water pressures tend to dissipate at a similar rate to the lowering water level so drawdown would then be unlikely to affect slope stability.

A popular rule of thumb (BRE 1999, Section 4.2.6) suggests the maximum rate of drawdown to avoid slope instability should be limited to 300mm/day, based on a typical 1V:3H upstream slope with fairly permeable shoulders. However, because the acceptable rate of drawdown is so dependent on the permeability of the slope and the embankment cross-section, it is better to calculate it by slope stability analysis based on the site-specific parameters. Some embankments such as head ponds at hydroelectric schemes regularly undergo drawdown rates of several metres an hour with no ill-effects, although these tend to be designed specifically for this purpose by sealing the upstream face (e.g. with asphalt or a watertight membrane) to prevent water permeating the fill.

In a real emergency, advice should be sought from an inspecting engineer regarding the relative importance of lowering the reservoir as quickly as possible versus maintaining upstream slope stability.

2.7 Proportion of full drawdown required

Partial drawdown of a reservoir is often sufficient to arrest an incipient failure, with full emptying of a reservoir often being difficult to achieve and not necessary. The depth of drawdown required to avert failure depends on the particular failure mode, with some typical examples given in Table 2.3.

As highlighted by Table 2.3, the depth to which it should be possible to lower a reservoir should be determined with consideration of the critical failure modes at that reservoir (see also Section 3.5). The failure mode which is likely to require the greatest depth of drawdown is frequently internal erosion, but again, the specific dam cross-section will govern where this threat is greatest and therefore the elevation to which drawdown would be particularly beneficial. A conservative assumption would be to consider internal erosion occurring through a concentrated leak at the base of the dam fill, although records of historical incidents suggest leaks often commence at the following locations:

- the point of maximum hydraulic gradient through the dam, which may not always be at the base (e.g. if there is a berm or changes in internal zoning)

- construction singularities (e.g. old crossing points over the core during construction, levels at which clay placing stopped one year and was resumed the next year, or historical interfaces from crest raising works)
- along the outside of conduits (e.g. low-level outlets, redundant pipework, cable ducts)
- through cracks caused by desiccation
- where there are changes in topography in the foundations (i.e. physical steps) which may cause differential settlement
- where there are changes in geology which may lead to preferential seepage paths
- along decaying tree roots
- through animal burrows

Table 2.3 Examples of magnitude of drawdown required to avert failure

Failure mode	Depth of drawdown required to arrest failure	Comments
Internal erosion	<p>There are several considerations:</p> <ul style="list-style-type: none"> • The depth to which the reservoir needs to be lowered depends on the elevation in the dam at which internal erosion is occurring (likely points of erosion are discussed below). • To arrest failure, the water level does not necessarily need to be lowered all the way to the depth at which erosion is occurring. Internal erosion is caused by the hydraulic shear stress of seepage flow acting on the soil particles, and there is a critical shear stress below which it ceases. Therefore the water level only needs to be lowered such that the seepage flow reduces to below this level of critical shear stress. At this point, the rate and turbidity of leakage should visibly reduce until the flow is clear. The critical shear stress varies with soil type and is discussed in Appendix D.2. 	<p>The emphasis of drawdown should be on preventing internal erosion from initiating once there are early signs of a potential failure mechanism. In some fill materials internal erosion appears to be relatively stable for long periods while for other dams with more erodible fill and/or high hydraulic gradients failure could be sufficiently rapid that no successful mitigating action is possible once internal erosion is sufficiently advanced.</p> <p>In order to make repairs following internal erosion, the water level is likely to need to be lowered below the point at which erosion has occurred.</p> <p>Note that drawdown is often used to stop leaks through the dam and enable repairs to take place, even when shear stress is below the critical level and therefore no visible internal erosion is occurring.</p>
Overflowing of dam crest causing erosion of the downstream face	Below the lowest elevation of the embankment crest.	This failure mode is likely to be triggered by a flood event and therefore is best managed by spillway provision rather than drawdown facilities. However, drawdown facilities may be beneficial in the event of spillway blockage in the aftermath of a flood event, or to hold the water level down to absorb subsequent floods without restarting erosion and to enable repairs.
Erosion of downstream face by spillway out of channel flows	Below the lowest elevation of the spillway weir crest.	Similar issues apply as above, although operating drawdown facilities is more likely to be beneficial in this case where modest flows down the spillway may result in substantial continuing damage.
Ice sheets causing horizontal loading and instability (concrete dams) or damaging valve towers and valve spindles on the upstream face	A moderate lowering in water level may cause the ice sheet to break up.	<p>For flexible structures such as embankment dams, horizontal loading from ice is not normally a major factor in stability calculations and, indeed, drawing down the reservoir can actually cause more damage in terms of displacement of rip rap on the upstream face (note 1) or damage to inclined valve spindles.</p> <p>Other mitigation measures such as bubbler systems may be better suited to address the risk of ice than drawdown.</p>
<p>Note 1. As the water level drops ice cover can crack 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.</p>		

As a general rule, it is always preferable to locate drawdown facilities as low as possible within the dam to provide the greatest depth of drawdown capability (and also because the higher head increases the rate of drawdown available). Justification for having drawdown facilities at a higher level should be based on a thorough failure mode assessment as described above.

The decision of how far to draw the reservoir down in an emergency should be based on advice by an inspecting engineer. The advice is likely to be to lower the water level until the symptoms of failure cease, and then lower it by a further amount to provide a margin of safety, for example to prevent flood inflows reinitiating the failure mode. In most cases, lowering a reservoir by one-third of its depth will significantly reduce the risk of failure progressing (this is equivalent to roughly halving the hydrostatic force).

2.8 Issues specific to flood storage reservoirs

In order to restrict flood flows, flood storage reservoirs employ various types of flow control devices ranging from simple orifices to more complicated devices such as hydro-brakes. These devices are normally fitted at the upstream end of culvert(s) passing through the dam with the culvert(s) typically being sized to pass flood flows during the construction period. For on-line flood storage areas, the control structure is commonly located at the point where the main watercourse passes through the dam. Trash screens are often installed to prevent the outlet getting blocked, but can sometimes make blockages worse if they are not regularly cleared.

Flood storage reservoirs may require slightly different considerations to conventional reservoirs due to their distinct characteristics, namely:

- They are generally empty for the majority of the time, which can increase the risk of desiccation cracking (i.e. the clay fill drying out and cracking). Such cracking can lead to leakage upon filling which could potentially cause internal erosion.
- Internal cracks, holes or deterioration may go undetected because all the time the reservoir is empty they would not be shown up by leakage.
- There may be an increased risk of trash washing down and blocking the outlets.
- By their nature, first filling of these reservoirs occurs during floods so compared to other reservoirs there may be an increased risk that an incident requiring emergency drawdown would be detected during a period of high inflows.
- Because flood storage reservoirs are only filled for short durations, the conditions for initiation and progression of internal erosion are less likely to develop during a single event, even with some relatively poor design aspects.
- Internal erosion at flood storage reservoirs is more likely where an existing structure (not originally designed as a dam) is being used as a flood storage dam.

Most flood storage reservoirs are designed to empty through the flow control device relatively quickly (i.e. within a few hours or days) after the flood has passed in order to be ready for the next flood. In some cases therefore the normal flow control device may provide sufficient capacity to empty a flood storage area in a reasonable time following the peak of the flood and additional drawdown capacity may not be necessary. However, in

some cases it is feasible for flood storage reservoirs to remain full for several weeks or even months at a time,² others may permanently store water below a certain level, and reservoirs performing in these ways are more likely to require additional drawdown capacity. Additional drawdown capacity is typically provided by means of a penstock gate either incorporated into, or adjacent to, the flow control device, discharging into the main culvert(s).

Depending on the outlet arrangement there may be a risk of the outlet becoming blocked by debris brought down with a flood or a trash screen becoming completely blinded by trash, and additional measures may be needed to mitigate these risks. For example trash screens may be adapted to ensure they can be cleared, or they may be designed to collapse under a certain head. Occasionally trash screens may be fitted with a by-pass pipe, to by-pass flow around both the trash screen and the flow control device, allowing the reservoir to be emptied without needing divers to clear the blockage (since this is potentially a very dangerous task).

Section 2.6 describes the effect of rapid drawdown on upstream slope stability. This may be less of a concern for flood storage reservoirs as the reservoirs may not be full for long enough for steady-state pore pressures to establish in the soil.

Guidance on design, operation and maintenance of flood storage areas is available in an Environment Agency publication (Environment Agency, 2016).

² For example Curry Moor flood storage reservoir in Somerset was full for around 3 months during the winter 2013/14 floods, although this is a fairly unusual example as retained water has to be pumped out into a tidal watercourse.

3 Site characterisation

3.1 Introduction

When evaluating the reliability and capacity of the existing and recommended drawdown capacity it is important to identify the site-specific constraints and characteristics relevant to drawdown reliability.

3.2 Review of available drawdown facilities

The first step in assessing drawdown capacity is to review the existing drawdown facilities available including the type, dimensions, elevation, condition and reliability of operation. Information may be obtained from past reports, record drawings or a physical survey, as deemed appropriate.

3.3 Assessing reservoir inflows

In order to make allowance for inflows into the reservoir during the drawdown period, it is necessary to assess the typical flows from incoming watercourses. The choice of inflow will affect the certainty of successfully being able to lower the reservoir and this is discussed in Section 5.2.

The catchment characteristics for the reservoir such as catchment area and percentage run-off should initially be established. Flow statistics can be obtained by locating the nearest gauged catchment on the Centre for Ecology and Hydrology website: <http://www.ceh.ac.uk/data/nfa/data/search.html>. For each gauging station the website allows a flow duration curve to be downloaded similar to the example shown in Figure 3.1. Such curves can be used to derive statistical flows which are exceeded on average for a certain proportion of days in a typical year. For example the Q_{50} flow would be exceeded on average for 50% of days in a typical year.

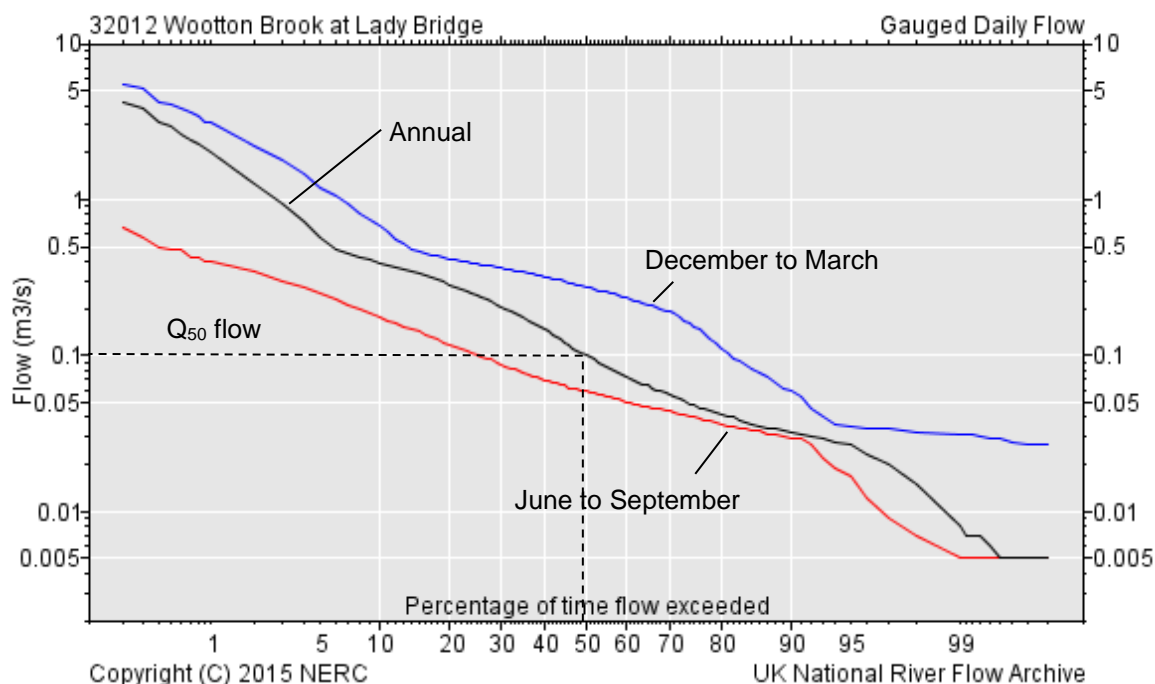
The flows obtained from the gauged data should be adjusted pro rata according to the catchment area of the gauging station (published on the website) relative to the catchment area of the reservoir. Care needs to be taken if the gauging station is a short distance downstream of the reservoir as the reservoir will act to attenuate flows and the outflow may be significantly less than the reservoir inflows. In these cases, and in cases where the reservoir catchment is ungauged, surrogate data may be obtained from a catchment of similar characteristics in the vicinity.

The above approach should provide a relatively quick estimate of inflows, sufficiently accurate for the purpose of calculating drawdown capacity in most situations.

A rule of thumb (Hinks 2009) suggests that for most areas of the UK, excluding Wales, the west of Scotland and perhaps the Lake District, the Q_{10} flow (i.e. that exceeded on average for 10% of days in a year) can be approximately estimated based on the catchment area as $Q_{10} = 0.035\text{m}^3/\text{s}/\text{km}^2$. Caution should be exercised when using this rule of thumb as it is based on limited analysis and is quite conservative in many cases.

Where a more precise estimate is required, a site-specific assessment should be carried out by a hydrologist. For example this approach may be appropriate if there is limited nearby gauged flow data and inflows are found to be a significant factor in the required drawdown capacity.

Figure 3.1 Example of how to obtain Q_{50} flows from a flow duration curve



Although not essential to determining drawdown capacity during an emergency it is often helpful to establish the peak flows and volumes of extreme floods, in order to assess the risk of the reservoir refilling while remedial works are carried out. Guidance on flood estimation is given in *Floods and reservoir safety*, 4th edition (ICE 2015).

3.4 Ability to divert or store inflows

3.4.1 By-wash channels

A by-wash channel is a channel cut into the valley side in order to convey water around a reservoir. They have a variety of purposes, including:

- diverting unwanted or polluted flows around the reservoir
- providing temporary flow diversion during dam construction or maintenance
- maintaining compensation flow or fish passage around the reservoir

By-wash channels often include control structures along their route to allow flows to either be diverted into the reservoir or by-passed around it. In most cases the capacity of a by-wash channel is within the range of typical flows from the catchment and they are rarely designed to take flood flows.

Where it is possible to divert reservoir inflows around the reservoir via a by-wash channel, the inflow pass-through allowance (see Figure 1.1) may be reduced accordingly. It is therefore necessary to assess the capacity of any by-wash channel. When assessing the capacity, the following items should be considered:

- What is the capacity of the by-wash channel that will be available in an emergency to divert reservoir inflows? It may serve other purposes as well so the full channel capacity may not always be available.
- What is the condition of the channel? Many by-wash channels are no longer maintained and have become derelict. If the channel is not routinely used and

maintained there is a risk it may be blocked when it is needed in an emergency, or that the banks may not withstand the flows discharged.

- If gates or valves need to be operated are they likely to be accessible in an emergency and will there be staff available to operate them within the assumed activation time (see Section 6.5.2)?

If reliance on a by-wash channel is assumed then it should be treated like any other aspect of a dam structure with an appropriate level of surveillance, inspection and maintenance and included in on-site plans.

3.4.2 Upstream reservoirs and aqueducts

The inflow pass-through allowance (see Figure 1.1) may also be reduced if there is the ability to store inflows, for example in an upstream reservoir, or divert them into another catchment. Again, if reliance on such structures is assumed then they should be treated with an appropriate level of surveillance, inspection and maintenance and be included in on-site plans.

3.5 Failure modes

Prior to commencing an assessment of drawdown capacity, it is important to consider the critical failure modes specific to the reservoir, using methods such as those set out in the *Guide to risk assessment for reservoir safety management (RARS)* (Environment Agency 2013), or other risk management publications. This requires information on the existing dam geometry, construction, foundation and outlets, and would normally require the same level of information provided for statutory inspections under UK reservoir legislation, such as drawings and previous reports.

Table 3.1 describes how drawdown could mitigate the risk of failure for a number of typical threats and failure modes.

Table 3.1 Relevance of drawdown capacity to potential failure modes

Threat	Potential damage and failure mode	How drawdown capability can mitigate the risk of failure (Note 1)
Deterioration (internal erosion)	Progressive erosion of fine material: from within the body of the dam/foundation, caused by concentrated seepage. The size of the leakage path can increase exponentially as the flow through it increases leading to an ultimate breach.	Drawdown would reduce the hydraulic gradient across the dam, reducing the rate of leakage and thus rate of erosion. Once the leakage rate reduces the shear stress below a critical value erosion would cease.
Floods	Structural failure of spillway: from high flow velocity.	Drawdown could be beneficial to mitigate failure modes developing following damage caused by a flood event. For example lowering the water level may ensure secondary floods do not cause further damage and reinitiate a failure mechanism.
	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 2013): from increased hydraulic loading.	

Threat	Potential damage and failure mode	How drawdown capability can mitigate the risk of failure (Note 1)
Wind	<p>Failure of erosion protection to the upstream face: Erosion of upstream fill by wave action resulting in localised slope failure and lowering of the crest, leading to overtopping (most likely during a subsequent flood event).</p>	<p>Drawdown could be beneficial to mitigate failure modes developing following wave damage as it removes the source of the damage. Lowering the water level may also ensure subsequent flood events do not cause further damage and reinitiate a failure mechanism.</p>
	<p>Large waves result in overtopping of the dam crest: Downstream slope failure (i.e. instability slip due to saturated fill and/or erosion damage) resulting in loss of freeboard then overflowing and erosion of the downstream face followed by back-cutting of the crest through which the contents of the reservoir can then flow.</p>	
	<p>Falling trees: Falling trees damage the dam or leave voids where they have uprooted, making the dam more prone to erosion by overtopping flows or other threats. Alternatively, fallen trees may cause the spillway to block.</p>	
Ice	<p>Upstream valve towers and spindles: Horizontal loads from ice produced by the thermal expansion of the ice sheet and by wind drag can result in significant forces which can damage valve towers or exposed spindles on the upstream dam face.</p> <p>Concrete/masonry dams: The horizontal loads can act to destabilise the dam.</p>	<p>Drawdown may assist in breaking up the ice and lowering the level at which the load is exerted.</p> <p>Other mitigation measures such as bubbler systems may be better suited to address this risk than drawdown.</p>
Earthquake (including liquefaction)	<p>Settlement of embankment and/or foundation: Lowering of crest and overtopping.</p>	<p>Drawdown could be beneficial to mitigate failure modes developing following earthquake damage (e.g. by removing load from the dam and reducing the likelihood of floods overtopping the crest).</p>
	<p>Slope failure: Degradation of slope, lowering of crest and overtopping, or; Increase in hydraulic gradient across core and internal erosion.</p>	
	<p>Disruption of filters: Reduction in filtering capacity and internal erosion.</p>	
Other threats including the actions of humans	<p>Examples include:</p> <ul style="list-style-type: none"> • terrorist actions • vandalism (e.g. shopping trolleys or other debris blocking spillways or outlet) • cutting down trees on the embankment, resulting in increased pore pressures and slope instability • excavation for services in the toe of the dam resulting in slope instability. 	<p>Drawdown could be beneficial to mitigate failure modes developing following such damage.</p>
<p>Note 1. In all cases drawdown would also generally reduce the consequences of failure and would be useful to allow repairs to be carried out.</p>		

For each failure mode, it may be useful to consider the various phases of progression (see, for example, Figure 8.11 of RARS) as the size of drawdown capacity required could depend on the phase during which it is mobilised. For example, the capacity to avert initiation will be smaller than the capacity needed to intervene after a major structural problem has developed.

3.6 Consequences of failure

The potential consequences of dam failure should be borne in mind throughout the assessment of drawdown capability, as a risk-based approach should assess the cost of any increase in drawdown capacity against the potential consequences of failure, and reduction in consequences that would be achieved. Reservoirs registered under UK reservoir legislation are given risk designations based on the potential consequences of them breaching, and impounding reservoirs are also categorised from A to D, in accordance with *Floods and reservoir safety*, 4th edition (ICE 2015). A more detailed assessment of potential consequences, including the likely loss of life (LLoL) may be required in some cases.

3.7 Other constraints on reliable drawdown capacity

Other site-specific constraints that may be important in terms of the effectiveness of drawdown capacity include:

- access to site
- access to valves once on site
- vulnerability of footbridges etc. to damage, or displacement caused by earthquakes, floods or waves
- other features such as by-wash channels or diversion pipes
- likelihood of ice formation
- environmental constraints (see Section 2.4)
- constraints due to the risk of localised downstream flooding (see Section 2.5)
- slope instability due to rapid drawdown (see Section 2.6)
- specific constraints associated with particular types of drawdown facility as detailed in Section 2.1

These should be catalogued when carrying out a review of drawdown capacity.

3.8 Surveillance and activation processes

Drawdown capacity needs to be based on the period available for drawdown, once the defect has been detected and the drawdown activated. Therefore the assessment of drawdown adequacy needs to take into account the likelihood that a defect would be detected in time for effective intervention, which depends on the frequency and effectiveness of routine surveillance, and the time it would take to activate drawdown. These topics are discussed further in Sections 6.5.1 and 6.5.2.

4 Determining installed discharge capacity

4.1 Hydraulic capacity

The maximum rate of discharge possible, with the reservoir initially at top water level, is termed the installed discharge capacity, Q_i . In some cases, this maximum capacity may not be reliably available and in such cases the reliable capacity should then be assessed as discussed in the following sections.

The rate of discharge depends on the reservoir head which will obviously reduce over the period of drawdown. For the purpose of calculating the installed discharge capacity, Q_i , it is appropriate to take the head at top water level and this is normally reasonably representative of the initial drawdown rate over the first 24 hours. For an accurate assessment of drawdown rate, particularly when considering drawdown over a longer period, it would be preferable to divide the reservoir into depth increments and use the proper head–discharge relationship and height–storage curve to calculate the time it would take to achieve each drawdown increment (as illustrated in Table 5.2). Although the discharge rate will reduce with falling reservoir levels, normally so too does the incremental storage volume of the reservoir and these two effects partially counteract each other when calculating the rate of drawdown as a depth per day.

Temporary and emergency drawdown capability should only be taken into account in the calculation of Q_i if an emergency plan exists to demonstrate it can be feasibly achieved within the necessary timeframe. Allowance should be made for the delay in installing any temporary facilities. Further guidance on this is given in Section 6.3.2. Discharge through operational draw-offs should only be taken into account if it is reliably available in an emergency situation (see Section 2.1.3).

For reservoirs registered under the Reservoirs Act 1975 it is a legal requirement that the maximum rate of discharge from any draw-off, bottom outlet or other permanent drawdown facility be documented in the reservoir's Prescribed Form of Record (in Part 8 of the original format, and in Part 6 of the new format). Similar requirements are made by reservoir legislation covering other parts of the UK. However, when assessing drawdown capacity, it is recommended that any values so documented are treated with caution and verified because experience suggests they can be incorrect. It should also be remembered that the values documented here are the maximum rate with the reservoir at top water level and do not take into account variations in the drawdown rate as the water levels lower as discussed above.

Methods to determine the hydraulic capacity of basic drawdown facilities are presented in Appendix B. In some cases it may be difficult to calculate Q_i due to complex arrangements of screens, gates, pipework and valves and it may be preferable to investigate the capacity by means of a site test. It is recommended that an experienced hydraulic engineer is consulted when calculating the existing hydraulic capacity.

4.2 Structural condition and reliability

Drawdown facilities should be maintained in good working order and regularly tested. Where this does not apply, then it may be appropriate to neglect the capacity of the dilapidated facilities in determining the existing drawdown capacity, or when carrying out a detailed risk assessment (see Section 8) the facilities should be assigned an appropriately reduced probability of availability.

Determining the site-specific structural condition and reliability of existing drawdown facilities is outside the scope of this guide, and reference should be made to CIRIA Report 170 (CIRIA 1997), CIRIA Report C743 (CIRIA 2015) and Lewin (2001).

4.3 Operational reliability

In addition to the physical reliability of the drawdown capacity, it is necessary to make a site-specific assessment of the likely operational reliability of drawdown capacity. In particular, operational reliability encompasses the following aspects:

- reliable access to the site to activate drawdown facilities, even at times of floods and bad weather
- reliable access to the valve controls themselves (e.g. if they require confined space access)
- the necessary trained staff being reliably available at short notice, at all times

Where the above aspects cannot be guaranteed then it may be appropriate to neglect the capacity of those drawdown facilities affected when determining the overall existing drawdown capacity.

Other guidance which may be helpful in evaluating operational reliability includes ICOLD Bulletin 164 (ICOLD 2013) and *Guidelines on dam safety management* (ANCOLD Incorporated 2003).

4.4 Reliable discharge capacity

The reliable discharge capacity, Q_R , should be assessed based on the hydraulic capacity of the installed drawdown facilities, Q_I , neglecting any facilities which may not be reliably available in an emergency either due to them being in poor structural condition, or for operational reasons.

5 Determining installed drawdown rate

5.1 Introduction

Section 4 provides guidance on how to calculate the *reliable discharge capacity* installed at a given reservoir. However, in the event that the reservoir needs to be drawn down, not all of this capacity will necessarily be available for emptying the stored water. Some of the capacity may be needed to pass inflows coming into the reservoir which would otherwise replenish the stored water and thwart the lowering effort. This concept is illustrated in Figure 1.1.

The discharge capacity available for reservoir lowering, Q_L is thus calculated as:

$$Q_L = Q_R - Q_P \quad \text{EQUATION 5.1}$$

Where:

$Q_L =$	<i>Discharge capacity available for reservoir lowering (m^3/s)</i>	<i>(The capacity to discharge stored reservoir water)</i>
$Q_R =$	<i>Reliable discharge capacity (m^3/s)</i>	<i>(The installed discharge capacity which can be reliably achieved) – see Section 4.4</i>
$Q_P =$	<i>Inflow pass-through allowance (m^3/s)</i>	<i>(The capacity required to discharge reservoir inflows which would otherwise replenish the stored water) – see Section 5.2</i>

The rate at which the reservoir water level can be lowered can then be determined based on Q_L and the reservoir volume, as discussed in Section 5.3.

5.2 Inflow pass-through allowance

As explained in Section 1.3, typical inflows into the reservoir (e.g. from incoming watercourses) need to be considered when assessing drawdown capacity. This component of the total drawdown capacity is termed the inflow pass-through allowance, Q_P (see Figure 1.1).

The magnitude of the inflow pass-through allowance needs to be balanced against the cost and practicability of installing larger drawdown facilities required for higher allowances. In most situations, the Q_{50} flow (i.e. the flow which is exceeded on 50% of days in a typical year) is considered to be an appropriate pass-through allowance to balance risk versus cost. Details of how to estimate the Q_{50} flow are given in Section 3.3.

Where it is possible to divert reservoir inflows away from the reservoir or store them upstream, the inflow pass-through allowance, Q_P , may be reduced accordingly. It is therefore necessary to assess the capacity of any by-wash channel, or other means of storage or diversion as discussed in Section 3.4.

The inflow pass-through allowance, Q_P should be determined as follows:

$$Q_P = Q_{50} - Q_X$$

EQUATION 5.2

Where: $Q_P =$ Inflow pass-through allowance (m^3/s)

$Q_{50} =$ Inflows exceeded 50% of days in a typical year (m^3/s)

$Q_X =$ Capacity of any by-wash channel, or other means of storing or diverting inflows (m^3/s) (see Section 3.4)

In addition, sensitivity checks should be carried out to understand how higher inflows such as the Q_{10} might hamper the ability to draw a reservoir down, because a reservoir may need to be drawn down, for either emergency or precautionary purposes, during a period of above-average inflows.

Table 5.1 describes examples of circumstances when an alternative inflow allowance to the Q_{50} flow might be appropriate.

Table 5.1 Examples of circumstances where the pass-through allowance may not be the Q_{50}

Issue	Details	Implications for Q_P
Non-impounding reservoir	For a non-impounding reservoir where there is full control over the reservoir inflows Q_P may be reduced to zero.	Q_P may be reduced to zero.
Consequences of dam failure (note 1)	Very high consequences associated with dam failure (e.g. certain category A dams where breach would cause a fast or deep flood wave affecting a large population). This does not apply to all reservoirs designated 'high-risk' under reservoir legislation, but just those where there are particularly high consequences.	This may suggest the need for a higher value of Q_P (e.g. up to the mean annual flood). It is recommended that in these cases an appropriate value of Q_P is determined based on a risk assessment process considering the combined probability of drawdown being required during any normal inflow condition.
	Low consequence of dam failure (e.g. reservoirs designated as 'not high-risk' under UK reservoir legislation).	Q_P should not normally be reduced below Q_{50} .
Upstream reservoirs	Typical outflows from any upstream reservoirs should be included in Q_P , unless they can be reliably controlled or diverted elsewhere, in which case they could be excluded. 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 routinely accommodated in the inflow pass-through allowance. Similarly the risk of an upstream dam break would not normally be considered. If upstream reservoirs offer the ability to attenuate inflows.	Could reduce Q_P if the upstream reservoir allows inflows to be attenuated or otherwise controlled or diverted.

Indirect catchment	Flows from any indirect catchments (e.g. flows from any pipelines or aqueducts bringing water into the reservoir) do not need to be considered in the inflow pass-through allowance provided a reliable emergency plan exists for diverting the indirect flows away from the reservoir within the assumed activation time (see Section 6.5.2).	No need to consider indirect flows provided they can be turned away from the reservoir.
High risk of snow melt	Occasionally reservoirs in upland catchments may be subject to high inflows at the end of winter from snow melt, which may create a significant spike on the annual hydrograph for the catchment. Flow from snow melt will be taken into account in the Q_{50} value obtained from gauged flow data and flow would not normally need to be considered separately. However, snow melt flows have caused issues with reservoir drawdown capability in the past (e.g. Rhymney Bridge Reservoir; Hughes and Williamson 2014) and some reservoir owners may wish to design for higher inflows than the Q_{50} to account for this.	Already taken into account in Q_{50} flow but some reservoir owners may wish to make an additional allowance.
Large reservoirs where drawdown would be prolonged	If drawdown is expected to take place over an extended period of more than just a few days then the Q_{50} value for a period of say 1 week or 1 month could be different.	If necessary, an experienced hydrologist should be consulted.
Note 1. Dam categories relate to Table 2.1 of <i>Floods and reservoir safety</i> (ICE 2015).		

5.3 Calculating the installed drawdown rate

5.3.1 Installed drawdown rate

The rate at which a reservoir can be initially lowered, after making a reasonable allowance for concurrent inflows into the reservoir, is termed the installed drawdown rate, D_I . This can be calculated using Equation 5.3.

$$D_I = \frac{86,400 Q_L}{A H} \times 100\% \quad \text{EQUATION 5.3}$$

Where: D_I = Installed drawdown rate (% of maximum reservoir depth, H , in 24 hours, hereafter abbreviated to %H/day)

Q_L = Discharge capacity available for reservoir lowering (m^3/s)

H = Maximum reservoir depth, taken as the difference from reservoir top water level to lowest ground level at the downstream toe (m)

A = Reservoir surface area at top water level (m^2)

Equation 5.3 simply multiplies the discharge capacity by the number of seconds in a day to estimate the volume of water that can be emptied in 24 hours of drawdown. Then, conservatively assuming the reservoir sides are vertical, it equates this volume to a depth. As discussed in Section 4.1 the value of Q_L may be simplistically taken based on the reservoir head at top water level, and similarly D_I may be based on the surface water area at top water level. However, in cases where the initial rate of drawdown is particularly high it would be more accurate to refine the method to take into account the site-specific height–storage relationship and the head–discharge relationship for the outlet(s), similar to the approach shown in Table 5.2.

The achievable drawdown depth is expressed as a proportion of the maximum reservoir depth, H . This is useful when reviewing the adequacy in Section 6, because it allows dam height to be eliminated as a variable.

5.3.2 Time to empty a significant portion of the reservoir depth

Although the installed drawdown rate, D_i , is generally considered the most critical parameter for measuring drawdown capacity, the time to empty a significant portion of the reservoir depth should also be considered as part of a comprehensive evaluation, and is particularly relevant where significant reliance is placed on mobile pumps. The choice of depth should be based on any specific level(s) associated with critical failure modes (see Section 2.7) but in the absence of such considerations it is recommended that the time it would take to empty the upper third of the reservoir depth, $T_{33\%}$, should be evaluated.

As a general rule of thumb, lowering the reservoir by this amount will roughly halve the hydrostatic load on the dam which should normally be sufficient to bring most failure mechanisms under control. For reservoirs in typical UK valleys somewhere between V-shaped and U-shaped, the top third of the reservoir depth normally contains roughly half of the total reservoir volume. For wide, fully bunded reservoirs the volume contained in the upper third of the depth may be less than half the total volume.

A tabular approach is recommended to calculate $T_{33\%}$, as illustrated in Table 5.2. This approach allows the discharge rate to be increased in phases, for example to take into account the activation time for mobilising pumps.

Table 5.2 Recommended calculation approach for determining $T_{33\%}$

A	B	C	D	E	F	G	H	I
Reservoir water level (m depth)	Reservoir volume below this depth (m ³)	Incremental volume (m ³)	Average head over outlet for this depth increment (m)	Average discharge rate (m ³ /s) (Note 2)	Inflow pass-through allowance, Q_P (m ³ /s)	Net outflow (m ³ /s)	Time to empty this depth increment (hours)	Cumulative time (days)
<i>TWL = top water level</i>			(Note 1)	<i>Look up from rating curve (e.g. Figure B.1)</i>	<i>See Section 5.2</i>	$= E - F$	$= (C / G) / 3,600$ <i>seconds in an hour</i>	
10 (TWL)	95,440	18,200	8.0	0.6	0.35	0.25	20.2	0.8
9	77,240	14,800	7.0	0.52	0.35	0.17	24.2	1.9
8	62,440	12,160	6.0	0.48	0.35	0.13	26.0	2.9
7	50,280	2,700	5.4	0.44	0.35	0.09	8.3	$T_{33\%} = 3.3$
6.7 (top third of depth)	47,580	8,100	4.9					
6	39,480	10,200	4.0					
5	29,280	8,320	3.0					
4	20,960	7,840	2.0					
3	13,120	6,240	1.0					
2	6,880	4,480	-					
1	2,400	2,400	-					

Notes

1. In this example the outlet is 1.5m above the reservoir bed.
2. The average discharge rate could include the phasing in of mobile pumps etc. taking into account mobilisation time, although this has not been included in this example.

6 Assessing the adequacy of installed drawdown rate

6.1 Applicability of this section

This section of the guide applies to embankment dams in the UK, including:

- homogeneous embankments
- zoned (clay core) embankments
- rockfill dams with a clay core

Some aspects of the methodology would need to be adapted to apply to other embankment dam types (e.g. where there is a watertight membrane or concrete face slab on the upstream face, or concrete/masonry walls within the cross-section). Each of these features may limit the maximum amount of seepage flow through the embankment and could mitigate the risk of internal erosion from initiating (see Section 6.4.3). Subject to an assessment of other failure modes, such features may therefore justify a lower drawdown capacity.

This section is not applicable to concrete dams and service reservoirs, which are considered in Section 7.

6.2 Outline of approach

This section outlines an approach for assessing the adequacy of the installed drawdown rate, D_I , and the time it would take to empty the upper third of the reservoir depth, $T_{33\%}$, for embankment dams. The method for determining D_I and $T_{33\%}$ is explained in the previous sections.

The adequacy should be judged based on a number of ‘considerations’ (see Table 6.1), which are discussed in more detail below.

Table 6.1 Considerations in assessing the approach

Consideration		Description
1	Basic recommended standard	A basic standard for minimum drawdown rate is recommended in Section 6.3, depending on the dam category. The recommended rates are based on certain assumptions and where these do not reflect the situation they may need to be adjusted (e.g. in accordance with guidance given on the other ‘considerations’ below).
2	Vulnerability of dam to rapid failure through internal erosion	It is important to understand how quickly internal erosion could progress to failure for a specific dam, and how drawdown could help to forestall the erosion process. This might give cause for amending the basic recommended standard.
3	Other factors	Other factors which may influence selection of an appropriate drawdown rate are discussed in Section 6.5.

Consideration		Description
4	Precedent practice	Comparison with precedent practice is discussed in Section 6.6. This is not an essential consideration but may form part of the overall assessment in some cases.

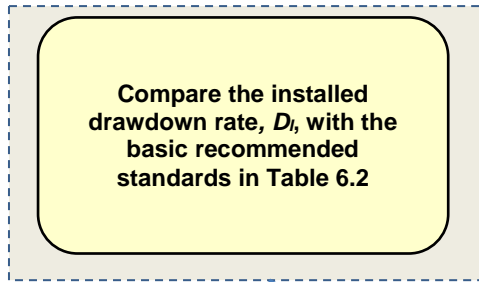
A flow chart outlining the approach is shown in Figure 6.1.

It should be emphasised that the adequacy of drawdown capacity needs to be assessed by an experienced engineer using engineering judgement to evaluate the 'considerations' outlined above in relation to a specific reservoir. For reservoirs registered under UK reservoir legislation the assessment should be reviewed by an inspecting engineer appointed under the legislation at the next Section 10 inspection. It is recommended that the evaluation is brought together and documented, with a possible tabular format proposed in Section 6.7.

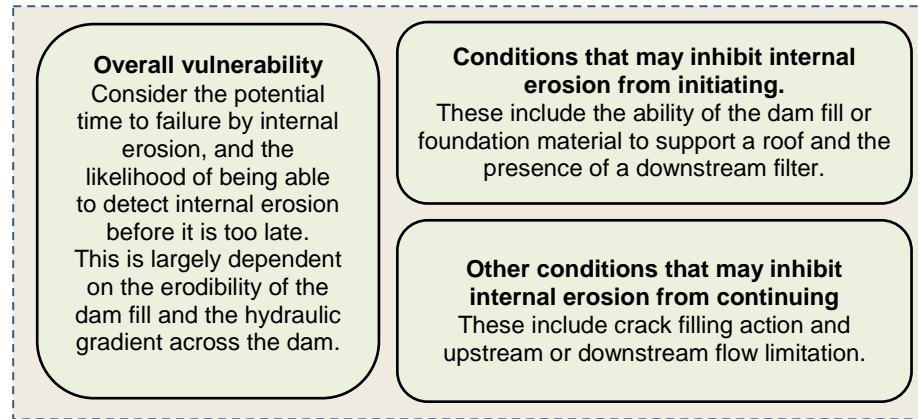
Where the existing drawdown rate is judged to be insufficient, then it is recommended that a risk-based approach is adopted to evaluate possible mitigation measures as discussed in Section 8.

Figure 6.1 Flow chart for assessing the adequacy of installed drawdown rate for embankment dams

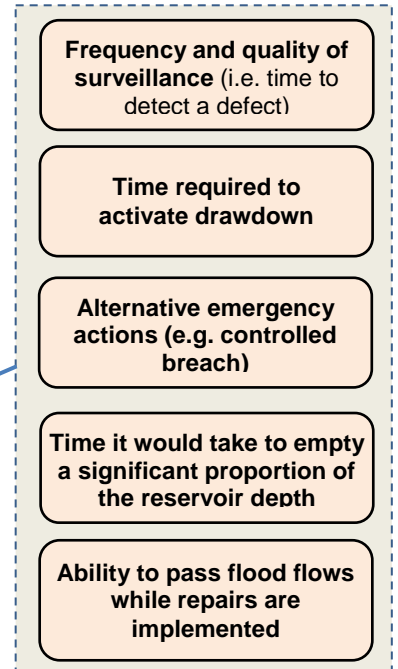
Consideration 1: Basic recommended standard (see Section 6.3)



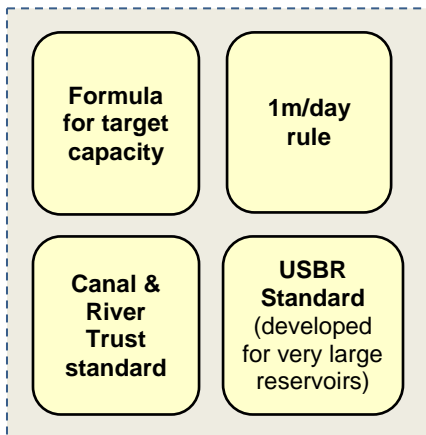
Consideration 2: Vulnerability to rapid dam failure (see Section 6.4)



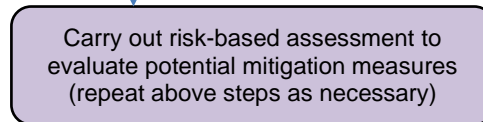
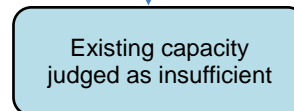
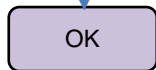
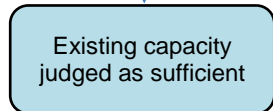
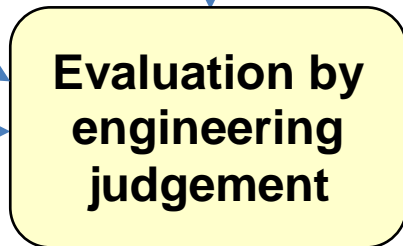
Consideration 3: Other factors (see Section 6.5)



Consideration 4: Precedent practice (see Section 6.6)



(Note this consideration may be omitted if desired)



(see Section 8)

6.3 Consideration 1: Basic recommended standard

6.3.1 Basic standard for drawdown rate

Basic standards for the minimum installed drawdown rate, D_i (see Section 5.3.1), are recommended in Table 6.2.

This ‘standards-based’ approach is similar to the long-established guidance for assessing spillway capacity in the UK (ICE 2015), with varying standards depending on the potential consequences of failure. A consultative process has been followed, including trials carried out on a sample of UK reservoirs, to ensure the values recommended, for the assumptions adopted, are generally acceptable within the profession.

The guidance in Table 6.2 uses the same four definitions of dam category (i.e. potential effect of a dam breach) as given in *Floods and reservoir safety* (ICE 2015). Higher drawdown capacity is recommended for dams where failure would threaten lives. For dams where there would be negligible risk to life (i.e. category C or D dams), the need for drawdown capacity should be assessed based on the reputational and economic costs of failure, in terms of damage to the reservoir asset and incremental flood damage downstream (which is also the liability of the undertaker). In some cases where there is limited risk there may be no need for any permanently installed drawdown capacity as, depending on the activation time, it may be acceptable to rely solely on temporary pumps or other emergency measures (see Section 6.3.2).

The recommended drawdown rates in Table 6.2 are expressed as a percentage of the maximum retained reservoir depth, H , which simplifies the table by eliminating dam height as a variable. However, such rates become impractical to achieve for dams and the table therefore allows the drawdown rate to be capped for this reason for higher dams over 20m in height. The cap is justified on the basis that such dams tend to conform to higher standards of design, construction and general management, but where this is not considered to be the case then application of the cap should be reviewed.

The recommended rates shown in Table 6.2 were derived following research, carried out as part of this project, on the potential time it would take typical dams to fail by internal erosion (see Appendix D). The recommended rates are also consistent with rates which have averted failure in actual drawdown incidents, based on the limited information available. The recommended rates shown in Table 6.2 are based on a number of assumptions as follows:

- 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. The vulnerability of a dam to rapid failure should be assessed separately as detailed in Section 6.4 and this may result in a decision to adopt a different drawdown rate.
- Good surveillance practices are employed (see definition of ‘good’ in Section 6.5.1).
- Drawdown can be activated shortly after a defect is detected (see Section 6.5.2). Further guidance on the reliance on temporary and emergency facilities is given in Section 6.3.2.

Where the circumstances differ from these assumptions, then the adopted drawdown rates should be reviewed based on the guidance in later parts of this section (i.e. Considerations 2 and 3). The guidance below is not statutory; however, it is

recommended that where an engineer feels it is right to depart from the rates provided, then reasons should be presented in support of that decision.

Table 6.2 Basic recommended standard for drawdown rate, D_1

Dam category (Note 1)	Recommended minimum rate (Note 2)	Upper cap on practical drawdown rate (Note 3)
A (Note 4)	5%H/day (Note 5)	1m/day
B	3%H/day (Note 5)	0.6m/day
C or D (Note 6)	2%H/day	0.3m/day

Notes:

1. The dam category is defined in *Floods and reservoir safety* (ICE 2015) based on the potential to life and damage downstream if the dam were to fail. Category C dams are those where there would be negligible risk to life and category D dams are those where no loss of life could be foreseen.
2. The rates are based on drawing the reservoir down from top water level.
3. The cap is considered justifiable on the basis that higher dams tend to conform to higher standards of design, construction and general management but where this is not considered the case then it may be appropriate to raise the cap.
4. Category A dams are those 'where a breach could endanger lives in a community'. For particularly large communities, e.g. where the likely loss of life (LLoL) exceeds 100 people, consideration could be given to increasing the recommended rates to drawdown from that shown above.
5. For low height dams where there is a risk to life the drawdown rate should be a minimum of 300mm/day unless there are alternative emergency actions which could be implemented to mitigate the risk.
6. For category C or D dams the recommended standard is based on protecting the value of the dam as an asset and avoiding potential reputational losses which may be associated with dam failure. Departure from the recommended standard could be considered if these potential losses can be tolerated.

6.3.2 Reliance on temporary and emergency facilities

Temporary drawdown facilities, such as mobile pumps and siphons, and emergency methods of drawdown are described in Section 2.1.4. Because they require significant advance planning, take longer to activate and generally provide a much smaller drawdown capacity than permanent facilities, temporary and emergency options are generally best used as a supplement to fixed drawdown facilities.

If reliance is to be made on temporary and emergency options the following standards apply:

- The activation time shall be taken into account. Further guidance on considering activation time is given in Section 6.5.2.
- The rates in Table 6.2 may be considered as average rates to empty a significant portion of the reservoir depth (see Section 6.5.4).
- Temporary and emergency drawdown capacity is only likely to be reliable if an emergency plan exists to demonstrate the capacity can be feasibly achieved within the necessary timeframe.
- For dams where there is a potential risk to life (i.e. category A and B dams as defined in ICE 2015), it would be reasonable for temporary and emergency drawdown capacity to not make up more than 50% of the total capacity deemed necessary.

In terms of the risk to public safety, it may be appropriate to have no permanent drawdown facilities and to rely solely on temporary and emergency facilities in the following circumstances:

- Small capacity reservoirs where the preferred drawdown capacity determined in Section 5 of this guide can be adequately achieved with temporary facilities, and where emergency planning indicates these facilities can be mobilised and set up quickly and reliably.
- Where the consequences of dam failure are low (i.e. dam categories C or D as defined in *Floods and reservoir safety*, ICE 2015).
- Low height dams where the hydraulic gradient is sufficiently low that internal erosion is unlikely to progress to failure.
- Where the cost of installing permanent facilities is disproportionate to the reduction in risk they would generate (see Section 6.5).

Even though permanent facilities may not be required in terms of public safety, they may still be desirable in terms of preserving the reservoir asset and water supply, and protecting against the reputational loss associated with a dam breach.

6.4 Consideration 2: Vulnerability to rapid dam failure

6.4.1 Introduction

The ultimate function of any drawdown facility is to reduce the load on the dam in an emergency sufficiently quickly to arrest a failure mode which has already initiated, or is at high risk of initiating, and prevent it from developing. The ability to serve this function should therefore be an important consideration when assessing the adequacy of drawdown facilities.

The speed at which an embankment dam would fail (i.e. breach and release the stored water) is primarily related to the vulnerability of the fill materials to erosion. This could be by erosion of the surface materials or by internal erosion. There is a broad correlation of the susceptibility of different materials to both forms of erosion.

Models of internal erosion have been developed to provide an indication of the relative vulnerability of different embankment dams. It is recognised that the science supporting these models is immature and it is only appropriate to use the results to assess relative vulnerability and not place too much emphasis on any absolute values calculated.

In a number of situations current models suggest that once it has initiated internal erosion would progress too rapidly for drawdown to be a practical means of mitigation. In other cases experience suggests that internal erosion can be stable for prolonged periods but can accelerate rapidly if conditions change. It is widely accepted that where there is a risk of internal erosion, reducing the load by drawdown can be an effective means of mitigation, principally by forestalling initiation and continuation of the process.

The speed at which internal erosion may develop will vary based on a number of factors. The following sections identify the key parameters and where possible provide an indication of the relative impact on the time to failure. The principal references on which the evaluation is based are:

- The 'Internal Erosion Toolbox' (USBR 2008)
- *Risk management, best practices and risk methodology* (USBR 2015)

- ICOLD Bulletin 164, *Internal erosion of existing dams, levees and dikes, and their foundations* (ICOLD 2013)
- *Engineering guide to early detection of internal erosion* (Defra 2007)

6.4.2 Overall vulnerability

There are two key factors that affect the potential for internal erosion to initiate and the speed at which it progresses, namely:

- the hydraulic gradient through the dam, i
- the erodibility of the dam fill (i.e. the erosion rate index, I_{HET})

Guidance on determining these parameters is given in Appendix C. To provide an indication of the relative impact of differences in hydraulic gradient and embankment material, a simplified model has been constructed of the development of internal erosion.

Research on the potential time to failure was carried out during the preparation of this guide and is summarised in Appendix D. This includes a graph (Figure D.1) which can be used to obtain an indicative estimate of the theoretical drawdown rate which would be required to draw a reservoir down, in the time it would otherwise take for failure to occur, starting from the moment when concentrated leakage is likely to be first detectable. This theoretical rate is termed the theoretical drawdown rate to avert internal erosion, D_0 . This can be compared with the assumptions referred to in Section 6.3.1.

The limitations of the graph are discussed in Appendix D.1. One of the limitations is that it does not take into account the concept of critical shear stress (i.e. that erosion will only initiate once the shear stress generated by the flowing water exceeds a certain threshold). This means that, in many cases, concentrated leakage through dams can be steady without continuously eroding and widening the seepage hole, and therefore it would never actually lead to failure in the absence of other changes (i.e. steady-state seepage). Conversely in situations where the critical shear stress is exceeded the size of the leakage path can theoretically increase exponentially with the increasing leakage flow through it, and catastrophic dam failure could occur rapidly. One example of this occurring at a dam shortly after the end of construction is Teton (CIRIA 2014). An initial assessment of the critical leakage rate, Q_c , at which the critical shear stress would be exceeded and internal erosion would continue, is given in Appendix D.2.

While the values derived above should not be used on their own to determine an appropriate drawdown rate, they should be used in conjunction with the subsequent sections to assess whether any change from the values proposed in Table 6.2 would be appropriate and to assess if the time to empty a significant proportion of the reservoir is adequate (see Section 6.5.4). This is illustrated in the examples in Appendix E.

6.4.3 Conditions that may inhibit internal erosion initiating

The following conditions may prevent internal erosion from initiating and mean that the dam is less likely to be vulnerable to rapid failure.

Ability of dam fill or foundation material to support a roof

Research into internal erosion has shown that it is unlikely to initiate where the dam fill or foundation material is incapable of supporting a roof to an erosion pipe. Guidance on this is given in Section 8.2.1 of ICOLD (2013). Essentially, cohesive materials are likely to support a roof and non-cohesive materials will not. Most dams in the UK are comprised of fill material that would support a roof.

Presence of a filter

An important safeguard against internal erosion is the presence of a sand filter incorporated into the downstream shoulder of a dam. These are common in modern zoned earth embankments but are typically absent in older UK dams. Sometimes, however, the grading of the downstream shoulder material may provide a similar filtering effect.

The theory of filter design is covered in various well known references (e.g. USBR 2011) but in essence the filter is designed such that the pore space between sand particles is large enough to allow seepage to drain but small enough to prevent any fine soil particles being carried away by the seepage flow. In theory, a properly designed filter should therefore mitigate the risk of internal erosion; however, taking account of the uncertainties and variability in filter design and construction, the risk of internal erosion cannot be totally removed.

The assessment of drawdown adequacy should consider whether a filter is present and the quality of that filter. A properly designed filter would be one that has been designed using a recognised standard (e.g. USBR 2011) and constructed with good quality control, with the whole process overseen by an inspecting engineer under UK reservoir legislation. Where such a filter is known to be present, the risk of internal erosion is significantly reduced.

Where the quality of the filter is not fully understood then a judgement needs to be made whether its presence would reduce the potential risk.

6.4.4 Other conditions that may inhibit internal erosion continuing

The following conditions will not prevent internal erosion from initiating but may prevent it from continuing. If these factors are present the dam is less likely to be vulnerable to rapid failure.

Crack filling action

For some dams with a central and sloping earth core and rockfill or gravel shoulders, granular particles upstream of the core may get washed into a crack and seal against a downstream zone, stopping the erosion process before it causes dam failure. This is known as 'crack filling action'. Whether it will be effective is dependent on the compatibility of particle sizes of the granular soils upstream of the core and in the downstream zone. Further guidance on crack filling action is given in Section 8.2.2 of ICOLD (2013).

Upstream or downstream flow limitation

If leakage flows would be limited by high hydraulic losses through upstream or downstream zones then this may also prevent internal erosion from continuing. Possible examples include dams with a concrete face slab, core wall or a zone of relatively fine-grained granular material. Guidance on this is given in Section 8.2.3 of ICOLD (2013).

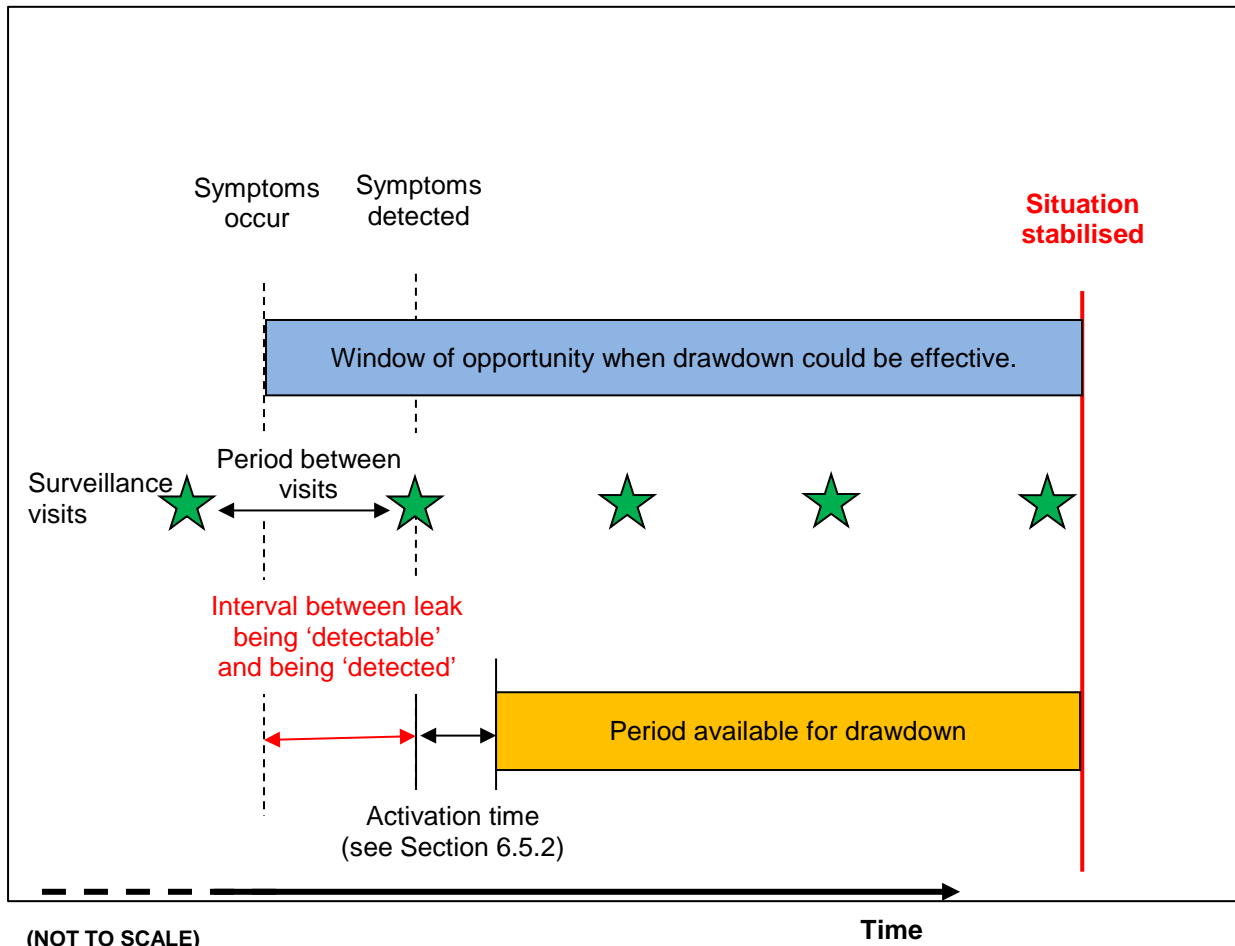
6.5 Consideration 3: Other factors

6.5.1 Frequency and quality of surveillance

On the basis that drawdown capacity should be sufficient to avert or arrest a failure mechanism, then the rate required depends on the window of time available between the

moment when symptoms (e.g. concentrated leakage, sink holes, damage or erosion) are detected and the point when drawdown has been effective. This time period represents the window of opportunity when drawdown could be effective. However, a proportion of this available window could be lost in the interval between the symptoms being 'detectable' and it actually being 'detected' during a routine surveillance visit. This concept is illustrated in Figure 6.2.

Figure 6.2 Timescale for drawdown



If symptoms are not detected early enough then it may be impossible to arrest the failure mode. Drawdown capacity needs to be based on the period available for drawdown, once the defect has been detected and drawdown commenced. Therefore the assessment of drawdown adequacy needs to take into account the frequency and effectiveness of routine surveillance. Examples of common surveillance practices are given in Table 6.3.

The reference position assumed in deriving the recommended values in Section 6.3 are based on good surveillance practices being employed. Where surveillance practices are anything less than 'good' then consideration should be given to increasing the adopted drawdown rate.

Table 6.3 Guide to benchmarking surveillance frequency and quality

Benchmark (note 1)	Surveillance procedure	
	Typical frequency of visits	Examples of surveillance practices
Very good	Every 2 days or less	<ul style="list-style-type: none"> • Grass length generally maintained less than 100mm long • Surveillance carried out along full length of downstream toe with good visibility of downstream face and any egress of seepage from culvert or tunnel • Measurement of seepage flows from any downstream toe drains • Surveillance carried out by trained personnel, ideally the same person each time, or if the person changes then they should be fully briefed regarding the 'normal' conditions • Use of a surveillance checklist • Regular public access (many incidents have been detected by members of the public such as fishermen or walkers and not the owner)
Good	Twice weekly	<ul style="list-style-type: none"> • Grass length generally maintained less than 200mm long • Surveillance carried out along full length of downstream toe with good visibility of downstream face and any egress of seepage from culvert or tunnel • Measurement of seepage flows from any downstream toe drains • Surveillance carried out by trained personnel, but not the same person each time • Use of a surveillance checklist • Occasional public access (many incidents have been detected by members of the public such as fishermen or walkers not the owner)
Medium	Weekly	<ul style="list-style-type: none"> • Embankment grass cover often exceeds 200mm length • Surveillance covers most of the length of the downstream toe or face • Visual check of seepage flows from any downstream toe drains • Surveillance personnel vary on each visit so changes are less likely to be noticed • No public access
Low	More than 1 week between visits	<ul style="list-style-type: none"> • Embankment grass cover regularly exceeds 300mm length or embankment surface obscured by other vegetation • Downstream face is only viewed from afar (e.g. the dam toe is not walked) • Toe drain covers are not regularly lifted • Surveillance carried out by untrained personnel • No public access
<p>Note 1. The benchmarks are relevant to category A dams. A reduced frequency is often appropriate for lower category dams.</p>		

Note that further advice on determining an appropriate surveillance regime is given in *An engineering guide to the safety of embankment dams in the UK* (BRE 1990).

6.5.2 Time required to activate drawdown

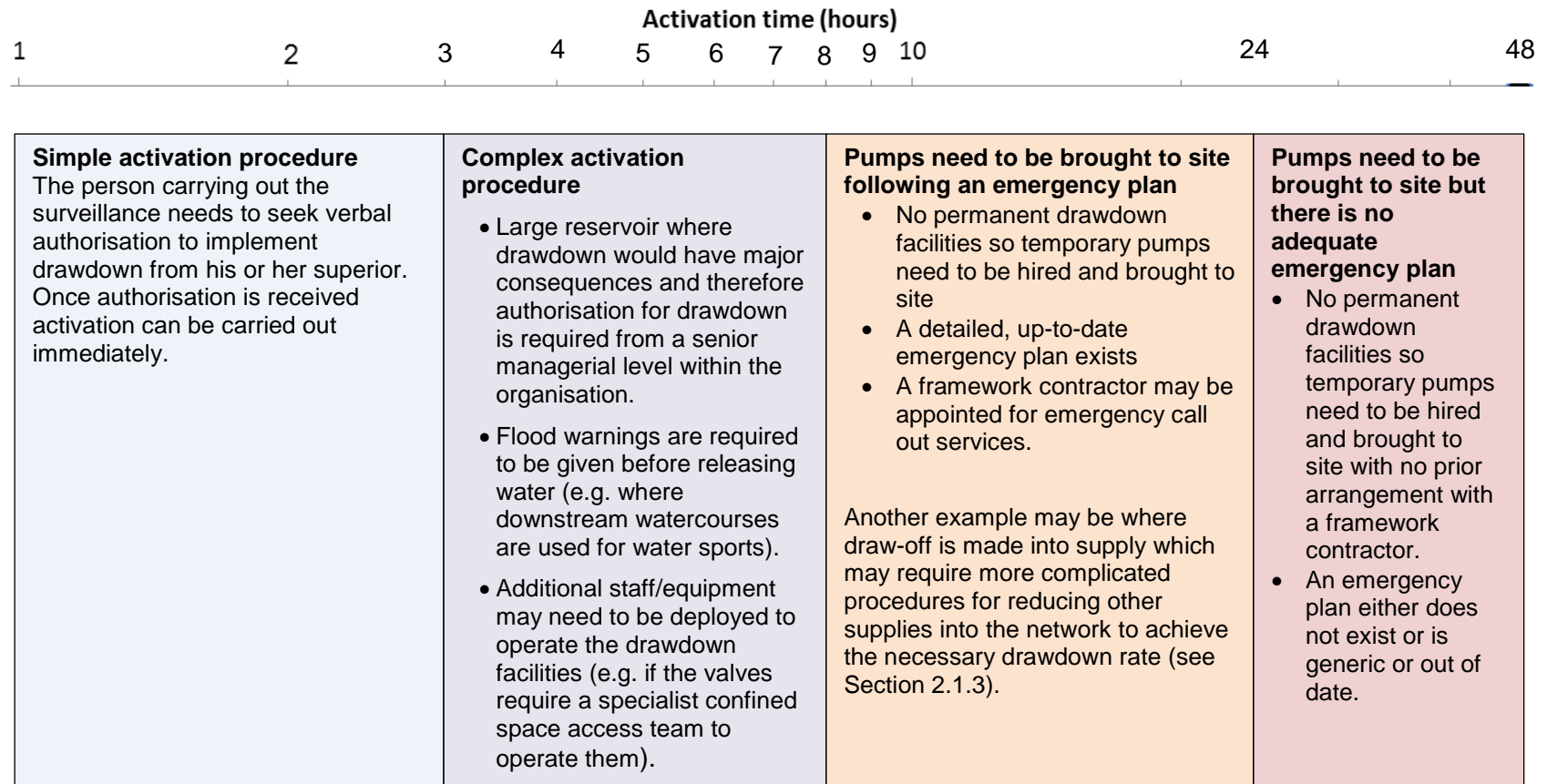
Once a structural problem has been detected there may be a period of time, known as the activation time, before drawdown can be commenced. This period may include some or all of the following activities:

- Reporting the observation from site, where there may be limited mobile phone signal.
- Escalating the issue to the appropriate decision maker within the reservoir owner's organisation.
- Making an assessment of the problem and a decision to lower the reservoir.
- Instructing the drawdown procedure.
- Implementing the reservoir drawdown which may require operatives travelling to the site, bringing pumps in and so on.

Activation time has a similar effect on drawdown capacity as surveillance frequency and is illustrated in Figure 6.3. The activation time will vary for specific reservoir owners and individual reservoirs.

The reference position assumed in deriving the recommended rates in Section 6.3 is based on drawdown being activated shortly after a defect is detected.

Figure 6.3 Examples of activation processes



6.5.3 Alternative emergency actions

It may be appropriate to accept a lower installed drawdown capacity where alternative emergency measures can be used to compensate, provided the feasibility and effectiveness of such actions has been properly assessed and agreed with an inspecting engineer, and planned in advance as part of the emergency plan. Examples of alternative actions include:

- a controlled emergency breach of the dam (see Section 2.1.4)
- upstream cofferdam or temporary raising of an upstream dam(s) to attenuate inflows
- evacuation of the downstream area
- emergency buttressing of the dam

6.5.4 Time to empty a significant portion of the reservoir depth

The recommended minimum drawdown rates given in Table 6.2 are based on the reservoir initially being at top water level. The rate is likely to vary in the longer term due to the reducing reservoir head, changes in the incremental storage volume with depth and possibly the phasing in of pumps or other measures.

A comprehensive assessment of drawdown capacity should therefore also consider the time it would take to empty a significant portion of the reservoir depth. The choice of depth should be based on any specific level(s) associated with critical failure modes (see Section 2.7) but in the absence of other considerations it is recommended that the time it would take to empty the top third of the reservoir depth, $T_{33\%}$, is evaluated as discussed in Section 5.3.2. Whether or not this time is deemed acceptable will depend on the vulnerability of the dam to rapid failure as discussed in Section 6.4. The surveillance frequency and activation time are also relevant as discussed above.

6.5.5 Ability to pass flood flows while repairs are implemented

If a reservoir does need to be drawn down in response to an incident, it may then need to be kept at a lowered level to enable repairs to be safely carried out. During this subsequent period the full capacity of the drawdown facilities will be available for controlling the reservoir level and passing inflows (i.e. with reference to Figure 1.1 there is no need for reservoir lowering capacity). However, the risk of flood inflows occurring during this period should be considered. If a serious fault has occurred at a dam it may take many months to design and implement repairs, and the longer it takes the greater the probability of floods occurring which could refill the reservoir and reinitiate the failure mechanism.

Consideration should be given to the duration repair works may take and what magnitude of floods should be designed for during this period. Guidance on an appropriate likelihood of flood to be considered is given in Chapter 7 of *Floods and reservoir safety* (ICE 2015).

Section 2.7 discusses the depth of drawdown required to prevent the failure mode from progressing. It would be reasonable to assume that, if repairs are going to take a long time, the reservoir could be lowered below this level to provide a 'buffer' of available reservoir storage which could be used to absorb floods without the water level rising to the point where it could cause the failure mechanism to reinitiate. Alternatively it may be feasible to construct an upstream cofferdam to divert or attenuate flood flows during the repair works.

The examples of particular circumstances described in Table 5.1 should also be considered when reviewing the flood risk in the aftermath of drawdown.

6.6 Consideration 4: Precedent practice

The reason that this guide was developed was because there was previously a wide disparity of approaches used to assess the adequacy of drawdown capacity in the UK. A review of published standards and practices was carried out as part of this study and is reported in Volume 2. It identified 11 standards or practices used in the UK and 5 further international standards. In most cases the rationale for the practices is not known and in some cases the rationale is not suitable for general use (e.g. several take no account of inflows). Excluding the lesser known approaches, and those that are relatively minor variations of the others, there are three primary systems regularly adopted in the UK as follows:

- formula for target capacity (Hinks 2009)
- 1m/day rule
- Canal & River Trust approach (Brown 2009)

In addition the United States Bureau of Reclamation's standard (USBR 1990) is probably the most widely used standard internationally although the reservoirs for which this was developed are very large reservoirs on large catchments of which there are very few in the UK.

These four approaches are described in the subsections below. On the basis that these approaches have been in use regularly over recent years, it is reasonable that they be taken into consideration as part of a comprehensive evaluation of drawdown rate, while acknowledging their rationale and limitations.

6.6.1 Formula for target capacity

This system was proposed by an inspecting engineer in a paper published in the *Dams and Reservoirs* journal (Hinks 2009). It gives a general formula for calculating the initial lowering rate in the first 24 hours as follows:

$$\text{Initial drawdown rate (mm/day)} = 300 + 5H + 8,640 Q_{10}/a$$

Where H = Height of the dam (m)

$$Q_{10} = \text{Inflows exceeded 10\% of days in a typical year (m}^3/\text{s)}$$

$$a = \text{Area of the reservoir (hectares)}$$

For most areas of the UK, excluding Wales, the west of Scotland and perhaps the Lake District, Hinks suggests that a typical value of Q_{10} can be approximated based on the catchment area as $0.035\text{m}^3/\text{s}/\text{km}^2$ (although see note of caution in Section 3.3). This enables the formula to be simplified to the 'English formula' as follows:

$$\text{Initial drawdown rate (mm/day)} = 300 + 5H + 300 A/a$$

Where A = Catchment area (km^2)

Where a reservoir has about 50% or more of the desirable capacity, the paper suggests that the deficit can sometimes be made up by temporary pumps subject to practical constraints (e.g. the time it would take for mobilisation).

The paper does not explain the rationale for the 300mm/day basic rate but it is understood that it was taken from Section 4.2.6 of the BRE guide to the safety of embankment dams

(BRE, 1999). The authors of that guide gave it as a typical maximum rate to ensure there is little risk of rapid drawdown stability failure for dams with fairly permeable shoulders and an upstream slope of 1 in 3.

The paper notes that the formula is not appropriate for large fully bunded reservoirs, such as those on the west side of London, because of the high hazard and thin puddle clay cores and it suggests these will often need up to 1m /day. It also suggests that for large flood storage reservoirs a lower drawdown capacity than that suggested by the formula may be appropriate.

6.6.2 1m/day rule

This policy was originally adopted by Thames Water, following a recommendation of an advisory panel of inspecting engineers, and states that drawdown capacity should be sufficient to lower the top metre of a reservoir within 24 hours (by fixed means). This rule (mentioned in Philpott et al. 2008) was tailored specifically to the characteristics of Thames Water’s larger reservoirs. These comprise zoned embankments with relatively thin clay cores with gravel supporting shoulders. The dam heights of the reservoirs concerned are in the range approximately 10 to 15m and the dams retain large stored volumes and are located in heavily built up areas. Since then the policy has been adopted by some other reservoir owners and panel engineers. The approach was commended by Defra in a letter issued to water companies in England (Defra 2002) as a factor in averting failure at a serious UK dam incident at that time, although it is noted this drawdown was achieved with supplementary pumping.

The rule makes no reference to concurrent inflows, because the reservoirs for which it was developed were non-impounding.

6.6.3 Canal & River Trust approach

This system (formerly known as the British Waterways approach) was published in the *Dams and Reservoirs* journal (Brown 2009). It states that drawdown capacity should be sufficient to empty 50% of the reservoir volume in a target number of days as shown in Table 6.4.

Table 6.4 Canal & River Trust approach

Overall consequence class (Note 1)	Number of days to lower the reservoir to 50% of volume when full, with inflow of winter daily mean flow	
	Surveillance once a week	Surveillance twice a week
A1	3 days	5 days
A2	5 days	7 days
B, C, D	7 days	9 days

Note 1. The consequence classes are as discussed in Section 6.3 except that category A dams are subdivided into two subcategories A1 and A2 (Brown and Gosden 2004). Category A1 dams are those where the likely loss of life would be 100 people or more.

The rationale for deriving the recommended targets is not stated. The approach allows the permanently installed drawdown capacity to be augmented with up to 1m³/s of temporary pumping capacity on the basis that a framework contractor is retained to provide this emergency call-out service within 24 hours. When evaluating the drawdown time, this delay of 24 hours before pumping capacity is available is included explicitly in the analysis.

6.6.4 US Bureau of Reclamation guide

Probably the most widely known international guide is that published by the United States Bureau of Reclamation (USBR 1990). This expresses the minimum drawdown capacity based on nine hazard and risk classes as shown in Table 6.5.

Table 6.5 Extract from USBR (1990)

Table 4.- General guide for determining emergency evacuation time (in days)									
Evacuation stage	High hazard, high risk	High hazard, significant risk	High hazard, low risk	Significant hazard, high risk	Significant hazard, significant risk	Significant hazard, low risk	Low hazard, high risk	Low hazard significant risk	Low hazard, low risk
75% Height*	10-20	20-30	30-40	20-30	30-40	40-50	40-50	50-60	60-90
50% Height*	30-40	40-50	50-60	40-50	50-60	60-70	60-70	70-90	90-120
10% Storage**	40-50	50-60	60-70	50-60	60-70	70-80	70-80	80-120	120-160
25% Height*	60-80	70-90	80-100	70-90	80-100	90-110	90-110	100-160	150-220

* Height is measured from the initial pool as defined in subsection A.3 of part I for determining evacuation requirements.

** Reservoir storage between original streambed and the initial reservoir water surface level as defined in section A.3 of part I for determining evacuation requirements.

This approach requires an allowance for concurrent inflows into the reservoir to be taken as the highest consecutive mean monthly inflows for the duration of the evacuation period.

It should be noted that when these drawdown rates are expressed as a percentage of maximum retained water depth to be lowered per day, this standard is significantly lower than most UK standards. For example assuming roughly 50% of the reservoir volume is contained at 75% height this standard would allow a time period of 30 to 40 days compared with 3 to 9 days recommended by the Canal & River Trust approach (Table 6.4).

The dams for which this standard is intended generally conform to modern design standards with embankment dams incorporating designed filters and constructed in the second half of the 20th century. The standard is intended for very large storage reservoirs on large catchments of which there are very few in the UK. It is probably not directly applicable to typical reservoirs in the UK where forms of construction, ground conditions and rainfall patterns are different.

6.7 Overall evaluation

As illustrated in Figure 6.1, the overall evaluation of whether the installed drawdown rate at a reservoir is adequate should be based on judgement by an experienced dam engineer taking into account the considerations described in Sections 6.3 to 6.6.

It is recommended that the approach is documented in a tabular format similar to that shown in Table 6.6. Two worked examples are given in Appendix E.

Table 6.6 Suggested format for documenting the evaluation of drawdown rate

Consideration		Ref. to section in this guide	Evaluation	Conclusion
Site characterisation				
Installed drawdown facilities		3.2	Briefly describe the specific-site issues which are relevant to drawdown.	Summarise if/how these issues may affect the evaluation
Reservoir inflows		3.3		
Ability to divert inflows		3.4		
Dam geometry, construction and foundation		6.3 & 6.4		
Failure modes		3.5		
Consequences of failure		3.6		
Constraints on drawdown capacity		3.7		
Surveillance and activation processes		3.8		
Determining installed drawdown rate				
Installed discharge capacity, Q_I		4.1	Rating curve for each installed drawdown facility (including planned temporary facilities).	State the calculated value of Q_I (m ³ /s)
Reliable capacity, Q_R		4.2 to 4.4	Describe any constraints that might affect the reliability.	State the assumed value of Q_R (m ³ /s)
Inflow pass-through allowance, Q_P		5.2	Describe the rationale for selecting Q_P . Also consider upper bound value of Q_P (e.g. based on Q_{10} inflows). Make allowance for any by-wash channel capacity or other means of storing or diverting inflows.	State the assumed value of Q_P (m ³ /s)
Discharge capacity for reservoir lowering, Q_L		5.1		State the calculated value of Q_L (m ³ /s)
Installed drawdown rate, D_I		5.3.1		State the calculated value of D_I (%H/day)
Time to empty the top third of the reservoir depth, $T_{33\%}$		5.3.2		State the calculated value of $T_{33\%}$ (days)
Assessing adequacy of installed drawdown rate				
Consideration 1: Basic recommended standard	Basic recommended standard	6.3	Compare D_I with the basic standard recommended in Table 6.2.	State the basic recommended rates derived from Table 6.2.
	Overview	Summarise how the installed capacity compares to the basic recommended standards.		
Consideration 2: Vulnerability to rapid dam failure	Overall vulnerability	6.4.2	Consider the potential for internal erosion to initiate and the speed at which it may progress. Consider the theoretical drawdown rate to avert internal erosion, D_0 , and the critical leakage rate, Q_c shown in Appendix D, relative to assumed values on which the basic recommended standard is based.	Assess whether any change from the values recommended in Table 6.2 would be appropriate.
	Conditions that may inhibit internal erosion initiating	6.4.3	Assess the likelihood of internal erosion initiating (e.g. details of any designed filter).	
	Conditions that may inhibit internal erosion continuing	6.4.4	Assess whether there are any factors that may prevent internal erosion from continuing.	
	Overview	Assess whether any change from the values recommended in Table 6.2 would be appropriate.		
Consideration 3: Other factors	Frequency and quality of surveillance	6.5.1	Summarise surveillance practices. If these are less than 'good' then additional drawdown provision may be warranted.	Assess whether any change from the values recommended in Table 6.2 would be appropriate.
	Time required to activate drawdown	6.5.2	Summarise activation process. If this drawdown cannot be commenced shortly after symptoms of failure are detected then additional drawdown provision may be warranted.	
	Alternative emergency actions	6.5.3	Consider whether any alternative emergency actions may be able to supplement or compensate for drawdown capacity.	
	Time to empty a significant portion of the reservoir depth	6.5.4	Consider the time it would take to empty a significant proportion of the reservoir and assess the adequacy based on the dam's vulnerability to rapid failure.	Conclude if this is acceptable or if additional provision is required.
	Ability to pass flood flows while repairs are implemented	6.5.3	Consider the duration after a drawdown event when reservoir levels might need to be controlled. Consider the magnitude and probability of flood that could be passed during this period by the reliable installed capacity.	
	Overview	Assess whether any change from the values recommended in Table 6.2 would be appropriate.		
Consideration 4: Precedent practice	Formula for target capacity	6.6.1	Consider how the installed drawdown rate compares with precedent practice.	Does this affect the previous assessment?
	1m/day rule	6.6.2		
	Canal & River Trust approach	6.6.3		
	USBR standard	6.6.4	Consider if this is applicable.	
	Overview	Summarise the adequacy of the installed drawdown rate based on the above considerations.		
Overall evaluation		Make an engineering judgement based on the various considerations above.		Conclude whether the installed drawdown rate is adequate or not. If it is insufficient, refer to Section 8.

7 Other dam types

7.1 Introduction

The majority of dams in the UK are earth embankments (earthfill dams accounted for nearly 80% of dams in the 1994 BRE Dam Register), and hence these form the basis of the guidance in the previous sections. However, concrete and masonry dams, and reinforced concrete service reservoirs are also numerous. Because they are constructed of generally non-erodible materials on non-erodible foundations, some of the failure mechanisms of embankment dams are not relevant to these dam types and the guidance set out in Section 6 will not necessarily apply. This section therefore gives guidance on how to determine an appropriate drawdown capacity for concrete and masonry dams, and reinforced concrete service reservoirs. A flow chart outlining the approach is shown in Figure 7.1.

7.2 Concrete and masonry dams

7.2.1 Specific types

Concrete and masonry dams are typically found in mountainous areas such as Wales and Scotland where rock is more abundant as a concrete or masonry dam construction material and for foundations. There are various types of concrete/masonry dam as detailed in Table 2.2 of CIRIA Report 148 (CIRIA 1996) but in the UK the majority are gravity or buttress dams for which the guidance below is based.

7.2.2 Previous guidance

Appendix 2 of CIRIA Report 148 gives some advice on the requirement for drawdown of concrete and masonry dams and notes that, being generally non-erodible, they are less likely to require rapid drawdown than earth dams. It recommends that the top 10% of the reservoir should be capable of being emptied reasonably quickly. It subsequently mentions a rate of 1/m per day under 'average' inflow conditions (i.e. presumably the Q_{50}) but this appears to be in the context of what might be achieved without compromising stability under drawdown.

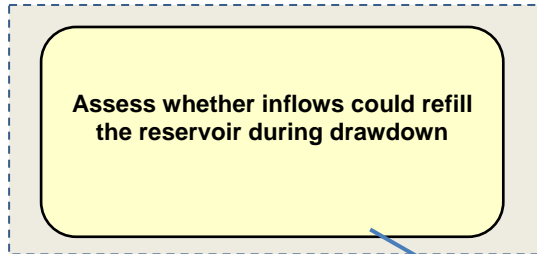
Most of the precedent approaches described in Section 6.6 are intended for embankment dams, so the consideration of precedent practice has not been included in the recommended approach for assessing the installed drawdown capacity (see Figure 7.1).

7.2.3 Failure modes

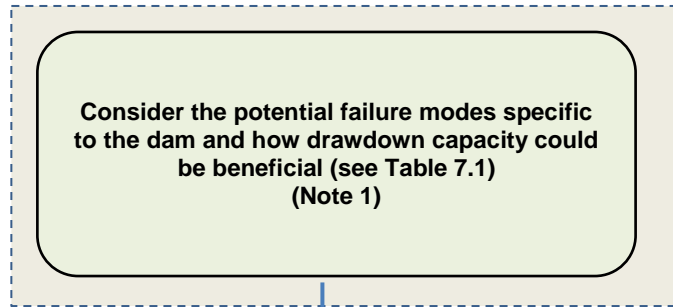
The failure modes which pose the greatest threat to concrete and masonry dams are summarised in Table 7.1, along with comments on how reservoir drawdown could be beneficial in an emergency.

Figure 7.1 Flow chart for assessing the adequacy of installed drawdown rate for concrete and masonry dams

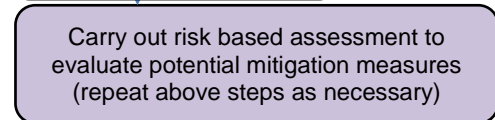
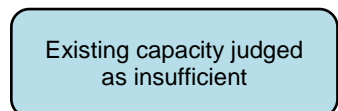
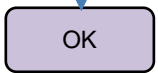
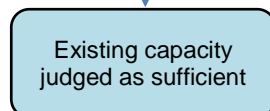
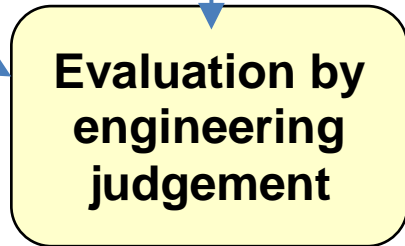
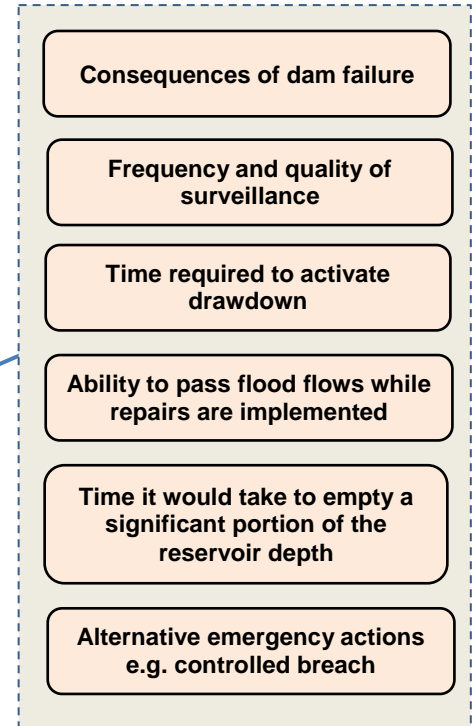
Consideration 1: Ability to achieve maintenance drawdown under a wide range of inflows



Consideration 2: Potential failure modes



Consideration 3: Other factors
(see Section 6.5)



(see Section 8)

Note.
1. Where any parts of the dam are erodible, (e.g. if sections of the dam are constructed on an earth foundation), then also consider the approach in Figure 6.1.

Table 7.1 Key failure modes relevant to concrete and masonry dams

Ultimate failure mode	Key factors	Possible causes	How drawdown capability could be beneficial
Sliding or overturning of the structure (global stability failure)	Increased uplift forces	Blockage of pressure relief drains	Dam movement would not be instantaneous and the problem should be identified by routine surveillance and dealt with before it becomes critical but drawdown may form part of the mitigation strategy . Drawdown may only need to be relatively gradual.
		High water level during floods	Failure during a flood would be brittle. As noted for embankment dams drawdown capacity is not intended to cope with flood events during normal operation.
	Increased horizontal forces	High water level during floods	As above.
		Ice loading (expansion of ice sheets or windblown ice floes)	Ice can exert significant forces but it will build up gradually over days or weeks and drawdown may be a useful tool for mitigating this risk (e.g. to assist in breaking up the ice and reducing the level at which the load is exerted) .
		Seismic forces	Failure would be rapid such that drawdown capacity is not relevant, other than allowing inspection and repairs after the event.
Scour at downstream toe	Overtopping	If there was scour damage at the downstream toe, drawdown could be used to lower the reservoir to reduce the risk of further overtopping .	
Uncontrolled leakage or water jetting	Cracks in the dam body or joints opening up	Degradation due to chemical reactions (e.g. alkali-silica reaction)	Generally slow processes which can be detected and managed through regular surveillance and maintenance. Drawdown may be required to prevent ultimate failure and allow repairs to be carried out , although it may only need to be relatively gradual.
	Deterioration of the foundation or grout curtain	Freeze-thaw action	
		Erosion of material from joints	
Failure of gates or other discharge equipment			Drawdown may be required to lower the reservoir to enable repair, although it may only need to be relatively gradual.

As noted in Table 7.1, failure modes for concrete and masonry dams tend to fall under two categories, global instability or general ageing and deterioration of the dam materials.

Global instability is often considered in terms of the factors of safety against sliding or overturning. In reality dams rarely fail by complete overturning because, as they start to rotate about the downstream toe, cracks develop at the upstream side increasing the uplift pressures and reducing the frictional resistance to sliding so that the ultimate failure mode tends to be

sliding. Once the forces acting on the dam exceed the restoring forces, then dam failure can occur very quickly, with the crack being almost instantaneous and the failure rate being governed by the failure block 'hanging up' at the release surfaces on either side. In these cases drawdown would not be an effective means of intervention and dam safety should instead be managed by ensuring suitable factors of safety against the design loading, and dam safety surveillance (e.g. by ensuring pressure relief wells do not become blocked and by carrying out regular monitoring of drain flows).

The second category of failure modes are slow processes of deterioration, which due to their nature would not warrant a particularly rapid drawdown capacity.

Perhaps the most critical failure scenario in terms of designing drawdown capacity is for moderate height dams which do not have well-designed pressure relief wells or means of monitoring uplift pressures. This is unlikely to be the case for dams over 5 to 7m height but may be true for dams lower than this. In these cases, the warning signs of failure are more likely to be missed, providing less time for reservoir drawdown.

In rare cases where any parts of the dam are erodible (e.g. if sections of the dam are constructed on an earth foundation), then internal erosion is likely to be a critical failure mode and it is recommended that the considerations in Section 6.4 are applied.

7.2.4 Purpose of drawdown

The main purpose of drawdown is similar to that for embankment dams as described in Section 1.2, that is to avert or arrest a failure mode which has already initiated, or is at high risk of initiating, and to prevent it from progressing. However, as discussed above, the failure mechanisms which apply to concrete and masonry dams are different to those for embankment dams (where they tend to be related to the erodibility of the fill material).

Based on the failure modes discussed in Section 7.2.3 it is assumed that the dam is still standing when drawdown of the reservoir is commenced. Hence, factors of safety for stability must be greater than 1.0 at that time, albeit possibly only by a small margin. On this basis, the purpose of drawdown should be to prevent water levels from rising and to also bring about a lowering of water levels, but such a lowering could be relatively gradual. This presumes that the dam is built on a non-erodible foundation; where this is not the case the considerations in Section 6.4 apply to ensure the load is reduced more rapidly.

Therefore the purpose of drawdown for concrete and masonry dams is to:

- remove the load from the dam to ensure an increase in the factor of safety for stability
- facilitate investigation and repair works
- provide a buffer to absorb floods without water levels rising sufficiently to cause a failure mode to be triggered

In the very worst outcome the intervention of drawdown may at least reduce the consequences of failure by reducing the volume of water released in a breach.

7.2.5 Assessing the adequacy of installed drawdown rate

The installed drawdown rate, D_I , and time to empty the top third of the reservoir volume, $T_{33\%}$, should be determined in the same way as for embankment dams following the guidance in Sections 3 to 5 of this guide. However, because the minimum recommended drawdown rates are less for concrete and masonry dams, the allowance for reservoir inflows is more likely to be a key factor in achieving drawdown and it is therefore even more important to carry out sensitivity studies for higher than average inflows.

The approach for assessing the adequacy of the installed drawdown rate should be judged based on the considerations shown in Table 7.2.

Table 7.2 Considerations for assessing the approach

Consideration		Description
1	Assess whether inflows could refill the reservoir during drawdown	<p>Although it may be acceptable to only have a slow rate of drawdown at concrete and masonry dams, it is important that water levels do not rise during the drawdown period as a result of high inflows. Checks should be carried out to ensure the risk of high inflows causing the reservoir water levels to rise is acceptably low.</p> <p>It is not the purpose of drawdown facilities to pass extreme floods but they may need to cater for higher than average inflows, with the design inflow dependent on the other considerations below.</p>
2	Potential failure modes	Consider the potential failure modes specific to the dam and how drawdown capacity could be beneficial (see Table 7.1).
3	Other factors	These are generally the same as for embankment dams (see Section 6.5) but also include the consequences of failure.

7.3 Service reservoirs

A service reservoir is defined by the Reservoirs Act 1975 (Statutory Instrument 1895, No. 1086) as 'a non-impounding reservoir which is constructed of brickwork, masonry, concrete or reinforced concrete'. They are enclosed structures incorporating a roof and are often partially buried below surrounding ground level. The maximum reservoir depth, H , should therefore be taken as the height above surrounding natural ground level. The perimeter walls often feature earthfill banks against the outer sides to structurally support the walls and/or provide a landscaping benefit.

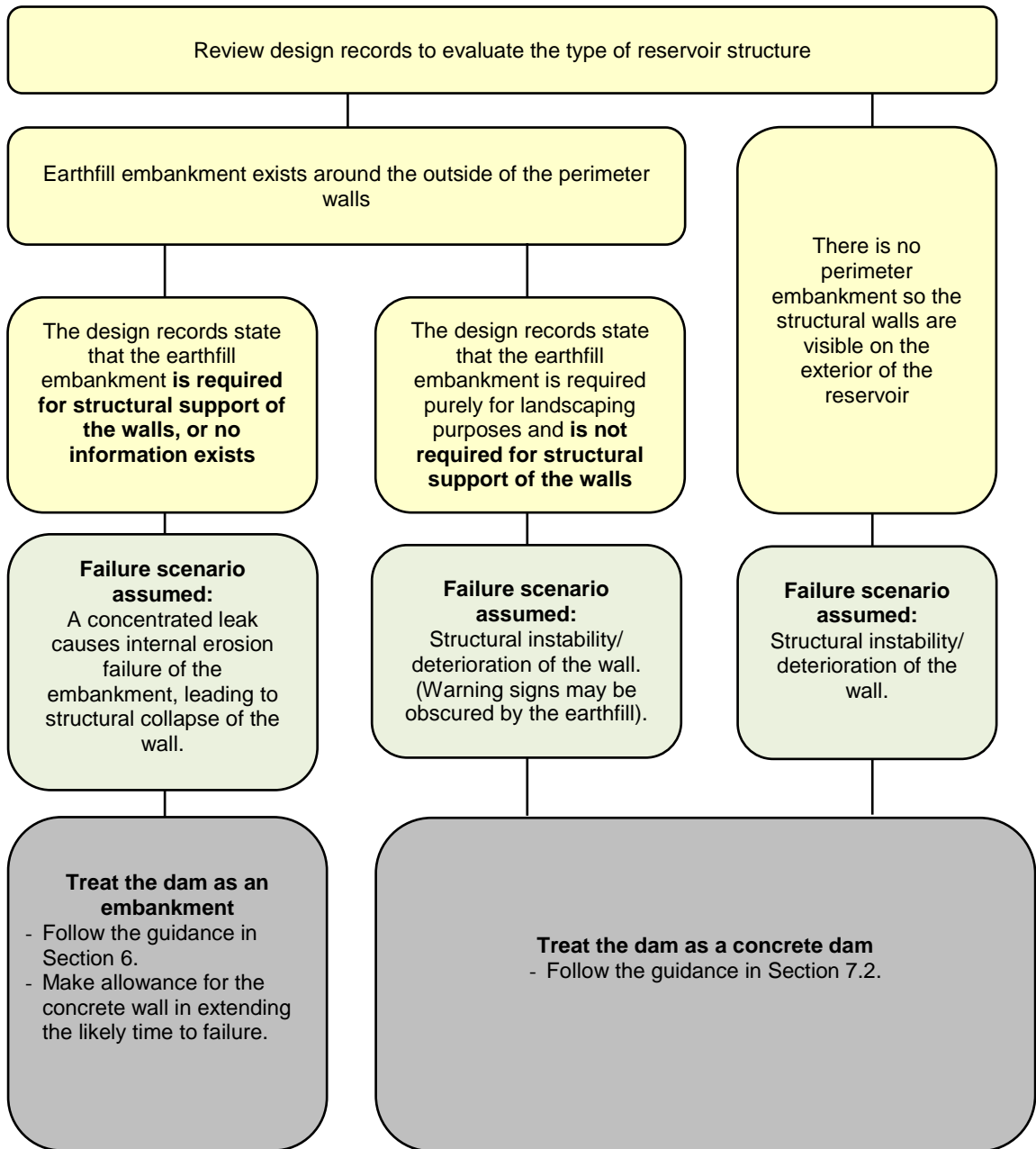
Service reservoirs therefore combine aspects of both concrete dams and embankment dams (if the earthfill banks are structural) and the method to determine an appropriate drawdown capacity should therefore be based on the respective approaches for each, depending on the precise arrangement. The approach is shown in Figure 7.2.

It is assumed in this guide that water levels are continuously monitored, with 'High' and 'High-High' alarms, and that inflows into the reservoir can be shut off if required in an emergency, and there is therefore no need to allow for pass-through capacity. Underdrains are an important aspect of surveillance at many service reservoirs which allow any leakage to be detected early.

In most cases the fastest means of drawing down a service reservoir will be into supply (see considerations in Section 2.1.3) but wash-out valves may provide some additional capacity and may be the only facility available if the failure mechanism has caused the reservoir water to become contaminated.

One major UK water company has a target standard to provide complete drain down of a single compartment within 12 hours. Achieving such drawdown rates can be difficult due to the fact that service reservoirs are often sited on hill tops where the receiving watercourses or surface water drains are often smallest. One option to overcome constraints in the capacity of the downstream channel or drain is to incorporate a spill chamber to attenuate the discharges.

Figure 7.2 Approach to determine appropriate drawdown capacity for service reservoirs



8 Mitigation against insufficient drawdown capacity

8.1 Introduction

If, following the evaluation described in Sections 6 or 7, the installed drawdown rate, D_I , is judged to be insufficient then mitigation measures should be considered to increase the effectiveness of reservoir drawdown, or if necessary to mitigate the risk of failure by other means. This section describes possible measures that could be adopted to mitigate against insufficient installed drawdown capacity. There is a wide range of options varying significantly in cost and effectiveness. It is therefore recommended that the most appropriate solution is identified using a risk-based approach so that the risks of not achieving drawdown when required are reduced to an acceptable level but disproportionately high costs are avoided.

The approach is similar to any normal feasibility study and should be overseen by an experienced dam engineer.

8.2 Options for mitigation

Options for mitigating against insufficient drawdown capacity broadly fall into four categories as follows:

- Increase the installed capacity.
- Increase the likelihood of detection of potential failure modes at an early stage to allow prompt intervention.
- Reduce the consequences of failure by improved emergency planning to ensure a potential failure incident can be effectively dealt with, or if the dam fails people downstream can be evacuated.
- Carry out improvement works to a dam in satisfactory condition, for example to reduce the likelihood of internal erosion occurring, or to slow the rate of progression, such that the installed drawdown capacity is judged adequate using the approach in Sections 6 or 7. Possible examples include:
 - reduce the hydraulic gradient (see Section 6.4.2 and Appendix C) either by carrying out structural works to the dam or lowering the water level
 - install a filter (see Section 6.4.3).

Table 8.1 gives examples of alternative mitigation options. There may also be other specific options at a particular reservoir. A solution may involve a combination of different options.

Table 8.1 Mitigation options

Mitigation option	How it helps	Examples
1. Improve or refurbish existing facilities		
Remove any restrictions within the existing outlet arrangement	Increases reliable discharge capacity	If the scour outlet is a small diameter branch off the main outlet pipe, the branch could be replaced with larger diameter pipework
Remove any constraints which restrict how much can be reliably discharged into operation or supply	Increases the drawdown capacity that can be reliably discharged into supply	See issues discussed in Section 2.1.3
Refurbish and institute proactive maintenance	Refurbishing the facilities may increase capacity in several ways (e.g. by increasing the amount valves can be opened or reducing frictional losses). Actuating valves may improve their reliability	Service and grease valves Reline outlet pipes
2. Increase installed drawdown capacity		
New low-level outlet	Provides additional discharge capacity	Drive a tunnel through the abutment
Enlarge the size of the existing outlet	Provides additional discharge capacity	Enlarge existing outlet pipes
Install drawdown capacity which avoids works at the base of the dam	Permits more rapid lowering of the upper part of the reservoir	Siphon Penstocks in spillway weir crest
Reduce inflows into the reservoir	Means that the existing drawdown facilities can be focused on drawing down the reservoir rather than passing inflows, such that the installed drawdown capacity is judged adequate using the approach in Section 6	Refurbish by-wash channel. Add controls to allow flows from indirect catchments to be turned away
3. Ensure structural problems would be detected early		
Increase frequency and/or quality of surveillance	Increases probability of detecting defects early, thereby allowing drawdown to commence earlier Increasing the frequency of visits may have staff time, cost and health and safety implications, so may not represent value for money	Increase visits Keep grass short to allow improved surveillance
Install instrumentation system to provide real-time monitoring	As above	V-notch weirs and telemetry to measure seepage in downstream toe drains/from culvert
Place signs at the reservoir asking members of the public to phone and notify of any unusual occurrences	Where there is public access on a dam it may be detected sooner by a member of the public	Incorporate as part of an existing interpretation board
4. Reduce activation time		
Improve access to site or valves, or actuate the valves	Reduces the time it would take to activate drawdown	Provide stoned access track to the dam Install valve spindle, or electric/hydraulic actuators to avoid the need for confined space access to operate valves
Replace temporary pumping capacity with a permanent facility	Removes mobilisation time associated with temporary pumps and improves certainty that pumps will be available	Standby diesel pumps or submersible pumps and hoses kept on site

Mitigation option	How it helps	Examples
Make provision for mobile pumps	Reduces the time it would take to install temporary pumps	Pump bases, provision for inlet and outlet hoses
Carry out practice exercises	Staff will be practised in the activation procedure Allows lessons to be learned	Various published examples (e.g. Brown et al. 2010, Windsor 2012)
5. Emergency planning – on-site		
Produce an on-site emergency plan	Increases the likelihood that effective actions will be implemented	Guidance on developing an on-site plan is available at: http://www.gov.uk/guidance/reservoirs-owner-and-operator-requirements#prepare-a-reservoir-flood-plan-and-flood-map
Arrange for pumps or siphons to be on standby so that they can be quickly mobilised	Increases the reliability that temporary facilities will be available	The Canal & River Trust have a framework contractor appointed to provide this service within 24 hours
Plan a means of controlled breach of low height section of the dam	This is another means to draw the reservoir level down in an emergency	Plan how the controlled breach would be formed, where an excavator could be hired from, and how it would access the site
6. Emergency planning – off-site		
Note that the responsibility for the preparation, ownership and instigation of off-site plans rests with the local authority		
Encourage and support local authority 'emergency planning officer' producing a dam specific off-site emergency plan	Reduces the time to implement the evacuation plan	
Encourage detailed study of evacuation routes or capacity	Identify potential blockages to effective evacuation of population downstream	
Allow practice, regular review or updates of off-site plan		
Encourage consultation with first community at risk	Reduce warning time/ improve effectiveness of evacuation	
Assist to identify critical infrastructure or services at risk and assist emergency planning officer to liaise with owners to identify risk	Reduces potential consequences of failure	Liaise with gas company to ensure gas mains could be shut down in the event of a breach

8.3 Risk-based assessment

This section gives guidance on evaluating mitigation options and assessing whether the cost of implementing a particular option is warranted by the reduction in risk it would achieve.

8.3.1 Qualitative versus quantitative approach

There are two approaches which can be taken for reservoir risk assessments:

- **Qualitative approach:** This is the simplest approach. It involves using judgement to evaluate the cost and practicality of implementing a particular mitigation option and balancing this against the benefits that would result in terms of reducing the risk of a reservoir failure.

- **Quantitative risk assessment (QRA):** This can be a more consistent method of assessment which requires estimating the reliability of intervention to avert dam failure for specific candidate upgrade schemes. The impacts and costs of failure also need to be quantified and an ALARP (as low as reasonably practicable) assessment would then be made to judge whether the cost of the candidate option is proportionate to the reduction in risk achieved.

A qualitative approach is significantly simpler to carry out than a quantitative assessment and is appropriate in many cases. A quantitative (ALARP) assessment is however recommended where the shortfall in desired drawdown capacity is large and the consequences of failure are high.

Both approaches are outlined in the subsequent sections, with references provided for more detailed guidance.

8.3.2 Qualitative approach

A qualitative risk assessment relies on the experience and judgement of the person making the assessment. It is therefore important that it is carried out by an appropriately experienced and qualified dam engineer, such as an inspecting engineer for reservoirs covered by UK legislation. Although it is not always necessary to develop detailed estimates of cost and probability, it is important that the costs and risks are broadly understood. For example, this may require references to typical costs from similar schemes or reference to inundation maps downstream.

The assessment should be documented to record the rationale.

8.3.3 Quantitative risk assessment

Overview

Guidelines on using this type of risk-based approach for reservoir safety management are given in the *Guide to risk assessment for reservoir safety management (RARS)* (Environment Agency 2013) and its predecessor the *Interim guide to quantitative risk assessment for UK reservoirs* (Brown and Gosden 2004). It is not the purpose of this document to replicate this existing guidance, but advice is given on the specific issues associated with applying QRA to the assessment of drawdown capacity.

The normal steps required in a QRA assessment are as follows:

- Assess the current probability of failure and the likely loss of life (LLoL) and plot these values on an FN-Chart.³ Assess whether the current societal risk is acceptable, unacceptable or within the range of tolerability (ALARP zone).
- Where the risk is in the ALARP zone it is necessary to evaluate whether the risk posed is acceptable, by evaluating whether the cost of mitigating the risk is grossly disproportionate to the reduction in risk that would be achieved. The proportionality should be assessed by calculating the cost to prevent a fatality (CPF) and comparing this with the value of preventing a statistical fatality (VPF).

³ An FN-Chart is given in the guidelines for tolerable risk provided in the Health and Safety Executive publication *Reducing risk, protecting people* (often referred to as R2P2), published in 2000, and as shown in Figure 9.2 of the RARS guide (Environment Agency 2013).

- The current published VPF by the Department of Transport for road and rail schemes is around £2 million. The basis for calculating the value of the risk is given in Section 10 of the RARS guide (Environment Agency 2013).
- The CPF can be calculated for each candidate option as follows:
 - Multiply the reduction in annual probability of failure by the LLoL.
 - Discount this over a 100-year appraisal period to give a present value of likely savings in lives, using a factor of 30 as recommended in Section 10.2 of RARS (Environment Agency 2013). Alternatively, it has been suggested that a factor of 57 may be more appropriate when considering the value of preventing a fatality over a 100-year planning horizon (see Table 6 of Brown et al. 2014).
 - Multiply the reduction in annual probability of failure by the potential cost of third party damages if the dam were to breach.
 - Again, discount this over a 100-year appraisal period to give a present value of the risk savings for third party damage, using a discount factor of 30 as above.
 - Calculate the CPF as the capital cost of the scheme minus the present value of the risk savings for third party damage, divided by the present value of likely savings in lives.
- The question arises as to when ALARP is satisfied. For the purposes of this guide a proportion factor (i.e. the ratio of CPF/VPF) of between 2 and 10 is recommended. The value selected should depend on the overall probability of failure, and the accuracy of the cost estimates, and assessments for LLoL and economic damage. Thus for a low probability of failure (say 1×10^{-6}) where accurate estimates are made, ALARP is judged to be satisfied (i.e. the upgrade is not warranted) if the CPF is $>£4$ million ($2 \times$ VPF); however, where the probability of failure is higher (say 1×10^{-4}) and the analysis is based on less accurate estimates, the CPF would need to be more like £20 million ($10 \times$ VPF) to justify not implementing the option.
- If the ALARP analysis indicates that the cost of mitigation is proportionate to the reduction in risk then the works should be implemented.

The challenge in using this approach in the case of drawdown capacity is that it is not straightforward to relate an increase in drawdown capacity to the probability of failure. This is discussed below.

Relevance of drawdown capacity to probability of failure of dam

The presence and capacity of a drawdown facility does not in itself create a likelihood of failure (other than along the outside of the outlet); that is, it is not a threat (direct cause of failure) or mechanism of failure (see Table 7.2 of RARS). Instead it affects the ability to avert failure when a structural problem develops.

An effective drawdown capacity can reduce the probability of failure due to some threats. One example is internal erosion, which takes (a variable length of) time to develop and if the hydraulic gradient can be reduced by lowering the water level, failure could be avoided. The presence of the drawdown capacity can be seen as affecting the probability of failure at the intervention stage in an event tree analysis (Phase 7 in Figure 8.11 of RARS).

Quantifying the reduction in probability of failure

The greatest challenge when carrying out an ALARP assessment to evaluate upgrades in drawdown capacity is to quantify how the upgrade would affect the overall probability of dam failure.

There are three potential approaches:

- Assigning standard index probabilities to the situations where there is no drawdown facility and the full recommended drawdown capacity.
- Adjusting the overall probability of failure to allow for drawdown capacity using factors inferred from historical failure rates.
- Assessing directly the current probability of failure of the reservoir and the effect increasing the drawdown capacity would have on the probability of failure. This is a more lengthy procedure.

Further details of these options are presented in Table 8.2, along with guidance on when each might be most suitable.

For initial assessment it is recommended that index probabilities are used as a means of comparing the relative benefits of drawdown capacity in terms of their potential to reduce the overall probability of dam failure. Index probabilities should be assigned for the following cases:

- no drawdown capacity available
- full drawdown capacity as judged to be appropriate using Sections 6 or 7 of this guide

The existing probability of failure can then be interpolated (using a log-linear relationship) based on the above values. The index probabilities are not an absolute measure of the overall probability of dam failure but are a metric for assessing the extent to which changes in drawdown capacity could impact it. There is no industry consensus on what values should be assigned for such index probabilities but several options are presented in Table 8.2.

The recommended approach is shown in the worked example in Appendix E. For the purpose of the worked example, Option A from Table 8.2 has been demonstrated but the method would be similar for the other options described.

The approaches shown in Table 8.2 apply to quantifying the reduction in probability of dam failure associated with mitigation options 1 to 5 in Table 8.1. They do not apply to mitigation option 6 (off-site emergency planning). This latter option would reduce the consequences of failure by reducing the fatality rate and thus the likely loss of life.

Table 8.2 Possible methods for deriving probabilities associated with drawdown capacity

Option		Description	Probabilities of failure (Note 1)		Guidance on usage
			No drawdown capacity	Full drawdown capacity (Notes 2 and 3)	
A Index probability methods (for screening)					
A1	Index probabilities proposed by Peters et al. (2016) (subsequently refined in Option A2 below)	One large water company has adopted the index probabilities shown in the next column. The values proposed for no drawdown capacity were selected on the basis of the societal risk of no outlet capacity being intolerable when average societal life loss (ASLL) was 10 or more, and allowing a factor of 0.1 times this for slow speed of failure. The value proposed for full drawdown capacity was taken as broadly acceptable for ASLL of 1,000, typical of high consequence dams.	Medium or fast speed of failure: 1 x 10 ⁻³ (1 in 1,000) Slow speed of failure: 1 x 10 ⁻⁴ (1 in 10,000)	1 x 10 ⁻⁷ (1 in 10,000,000)	This approach assumes that drawdown capacity could reduce the overall probability of dam failure by three or four orders of magnitude. It has been suggested by some that this may overestimate the benefits.
A2	Refinements to Option A1	Refinements to the original Peters et al.'s approach (Option A1) were proposed following the question and answer session at the British Dam Society Conference in Lancaster in September 2016 as published in the conference discussion (available on the conference page of the BDS Website).	As Option A above	Assign an index probability equivalent to having a spillway that can pass the probable maximum flood (PMF) as the safety check flood (i.e. annual probability of failure in the order of 1 in 400,000).	It could be argued that this is more defensible than the 1 in 10 million value quoted in the paper (Option A1 above).
A3	Postulation by engineering judgement	The index probabilities would be judged by an experienced reservoir inspecting engineer, based on his or her knowledge of the site and using more tangible probabilities such as flood return periods for comparison.	It has been suggested that the index probabilities might reasonably be expected to vary by say two orders of magnitude for a typical dam.		This allows inspecting engineers some freedom to adapt the other methods described above.
B Adjustment to probabilities inferred from historical failure rates					
B	Method given in the <i>Interim guide</i> (Brown and Gosden 2004)	The <i>Interim guide</i> (Brown and Gosden 2004) allowed for the outlet capacity (and reservoir operation) in the scoring of contributory factors to the current condition score for internal threats (Rows 35 to 39 in sheet 4.4 and similar in sheet 5.4; on pages 125 and 131 of the guide).	Calculate the probability of failure due to internal threats using the approach in Section 8 of RARS (Environment Agency 2013), and add adjustments for drawdown capacity as defined in the <i>Interim guide</i> .		It is considered that this approach may underestimate the benefits of drawdown capacity.

Option		Description	Probabilities of failure (Note 1)		Guidance on usage
			No drawdown capacity	Full drawdown capacity (Notes 2 and 3)	
		Where there was no outlet present it would increase the current condition score by one, broadly equivalent to doubling the probability of failure. It was noted that this was not included in RARS (Environment Agency 2013), which is understood to be an accidental, rather than a deliberate, omission.		The maximum impact of drawdown capacity would be to halve this probability of failure.	
C Direct probability method					
C	Use an event tree to evaluate the probabilities of failure for a range of scenarios, with effect of drawdown capacity as one of the considerations	This method would adopt the Tier 3 QRA approach given in RARS (Environment Agency 2013) to consider a range of potential failure scenarios using an event tree diagram. The approach would consider the conditional probability of individual factors such as concurrent inflow.		Impact of drawdown capacity on probability of failure to be judged based on the outcome of the event tree analysis.	This is considered the most accurate method but requires a detailed level of assessment.
Notes <ol style="list-style-type: none"> 1. For methods under A the existing probability of failure should be interpolated between this range, based on the percentage of the installed capacity relative to the recommended capacity. Note that the interpolation should be based on a log-linear relationship as shown in the example in Appendix E. 2. Full drawdown capacity is the capacity judged to be appropriate using Sections 6 or 7 of this guide. 3. For very high consequence dams, these low probabilities of failure become questionable and instead the safety case suggested in Brown and Hewitt (2016) may be more appropriate. 					

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Notation

Note that these symbols may have different meanings for the precedent methods described in Section 6.6. Terms used in Appendix B and Appendix C are defined there and are not repeated here.

<i>Symbol</i>	<i>Description</i>	<i>Reference to full definition</i>
A	Reservoir surface area at top water level (m ²)	5.3.1
D ₀	Theoretical drawdown rate to avert internal erosion	6.4.2 and Appendix D
D _i	Installed drawdown rate (%H/day)	5.3.1
H	Maximum reservoir depth retained by the dam, taken as the difference from top water level to lowest ground level at the downstream toe (m)	5.3.1
i	Hydraulic gradient through the dam	6.4.2 and Appendix C
I _{HET}	Erosion rate index of the dam fill or foundation material (measured from the hole erosion test)	6.4.2 and Appendix C
Q ₁₀	Inflows exceeded 10% of days in a typical year (m ³ /s)	5.2, 3.4
Q ₅₀	Inflows exceeded 50% of days in a typical year (m ³ /s)	5.2, 3.4
Q _X	Capacity of any by-wash channel, or other means of storing or diverting inflows (m ³ /s)	5.2
Q _C	Critical leakage rate at which internal erosion will continue	6.4.2 and Appendix D
Q _I	Installed discharge capacity (with the reservoir at top water level) (m ³ /s)	4.1
Q _L	Discharge capacity available for reservoir lowering (m ³ /s)	5.1
Q _P	Inflow pass-through allowance (m ³ /s)	5.2
Q _R	Reliable discharge capacity (neglecting any facilities which may not be reliably available in an emergency) (m ³ /s)	4.4
T _{33%}	Time it would take to empty the upper third of the reservoir depth (hours or days)	5.3.2

Appendix A List of equations

Equation no.	Equation	Where	Section in guide
5.1	$Q_L = Q_R - Q_P$	<p>$Q_L =$ Discharge capacity available for reservoir lowering (m^3/s) (The capacity to discharge stored reservoir water)</p> <p>$Q_R =$ Reliable discharge capacity (m^3/s) (The installed discharge capacity which can be reliably achieved) – see Section 4.4</p> <p>$Q_P =$ Inflow pass-through allowance (m^3/s) (The capacity required to discharge reservoir inflows which would otherwise replenish the stored water) – see Section 5.2</p>	5.1
5.2	$Q_P = Q_{50} - Q_X$	<p>$Q_P =$ Inflow pass-through allowance (m^3/s)</p> <p>$Q_{50} =$ Inflows exceeded 50% of days in a typical year (m^3/s)</p> <p>$Q_X =$ Capacity of any by-wash channel, or other means of storing or diverting inflows (m^3/s) (see Section 3.4)</p>	5.2
5.3	$D_1 = \frac{86,400 Q_L}{A H} \times 100\%$	<p>$D_1 =$ Installed drawdown rate (% of maximum reservoir depth, H, in 24hrs, hereafter abbreviated to %H/day)</p> <p>$Q_L =$ Discharge capacity available for reservoir lowering (m^3/s)</p> <p>$H =$ Maximum reservoir depth, taken as the difference from reservoir top water level to lowest ground level at the downstream toe (m)</p> <p>$A =$ Reservoir surface area at top water level (m^2)</p>	5.3
B1	$Q_{Orifice} = CA\sqrt{2gH}$	<p>$Q_{Orifice} =$ Discharge limited by entry into the orifice</p> <p>$C =$ Contraction coefficient (generally around 0.61 – See Chadwick et al. (2013, p.54))</p> <p>$A =$ Cross-sectional area of the outlet</p>	Appendix B

		<p>$H =$ Head measured from the reservoir water level to the centreline of the orifice</p> <p>$g =$ Acceleration due to gravity (9.81m/s^2)</p>	
B2	$Q_{Pipe} = A \sqrt{\frac{(2gH)}{\frac{fL}{D} + \sum K}}$	<p>$Q_{PIPE} =$ Discharge limited by pipe flow condition</p> <p>$H =$ Available hydraulic head, taken as the difference between top water level in the reservoir, and the water level downstream of the outfall pipe</p> <p>$f =$ Friction factor determined using the Moody chart in Figure B.2.</p> <p>$A =$ Cross-sectional area of pipe</p> <p>$L =$ Length of pipe</p> <p>$D =$ Pipe diameter</p> <p>$\sum K =$ Sum of local loss coefficients to allow for any 'disruptions' to the flow of water in the pipe, including entries and exit, bends, branches, valves, and contractions and expansions (see Table B.1)</p>	Appendix B
B3	$Z_s = \frac{P_a}{\gamma} - \frac{P_v}{\gamma} - \frac{V}{2g} - h_{fp}$	<p>$\frac{P_a}{\gamma} =$ Atmospheric head which is 10m</p> <p>$\frac{P_v}{\gamma} =$ Vapour pressure head which is 0.2m at 15°C</p> <p>$\frac{V^2}{2g} =$ Velocity head in the suction pipe</p> <p>$h_{fp} =$ Friction loss in the suction pipe</p>	Appendix B
C1	<p>Hydraulic gradient, $i = \frac{H}{L}$</p>	<p>$H =$ Maximum retained depth of water (m)</p> <p>$L =$ Effective seepage length, i.e. the maximum horizontal width of the water-retaining element of the embankment at the location of H (m)</p>	Appendix C
C2	<p>For coarse-grained soils:</p> $I_{HET} = 6.623 - 0.016\rho_d - 0.104 \frac{\rho_d}{\rho_{dmax}} - 0.044\omega - 0.074\Delta\omega_r + 0.113S + 0.061Clay(US)$	<p>$I_{HET} =$ Erosion rate index</p> <p>$\rho_d =$ Dry density of the soil (Mg/m^3)</p> <p>$\frac{\rho_d}{\rho_{dmax}} =$ Percentage compaction (%)</p> <p>$\omega =$ Water content (%)</p> <p>$\Delta\omega_r =$ Water content ratio relative to optimum moisture content, i.e.</p>	Appendix C

		$\Delta\omega_r = \frac{(\omega - OMC)}{OMC} (\%)$ <p><i>S</i> = Degree of saturation (%)</p> <p><i>Clay (US)</i> = Mass fraction finer than 0.005mm (%)</p>	
C3	<p><i>For fine-grained soils:</i></p> $I_{HET} = -10.201 + 9.572\rho_d - 0.042 \frac{\rho_d}{\rho_{dmax}} + 0.103\omega$ $+ 0.0097\Delta\omega_r - 0.0056Fines$ $+ 0.042Clay(US) - 0.090LL + 0.111I_p$ $+ 0.443Pinhole$	<p><i>Fines</i> = Fines content (<0.075mm) (%)</p> <p><i>LL</i> = Liquid limit (%)</p> <p><i>I_p</i> = Plasticity index (%)</p> <p><i>Pinhole</i> = Pinhole Test Classification expressed as an ordinal number, i.e. '1' for Class D1, '2' for Class D2, '3' for Class PD1, ..., '6' for Class ND1</p>	Appendix C
C4	$I_{HET} = -\log C_e$	<i>C_e</i> = Detachment/ erodibility coefficient (References: Hanson et al. 2010, ICOLD 2013)	Appendix C
C5	$C_e = k_d/\rho$	<i>k_d</i> = Detachment/erodibility coefficient (cm ³ /N/s) <i>ρ</i> = Dry density (Reference: Hanson et al. 2010)	Appendix C
C6	$k_d = \frac{10\gamma_w}{\gamma_d} \exp \left[-0.121(C\%)^{0.406} \left(\frac{\gamma_d}{\gamma_w} \right)^{3.10} \right]$	<i>k_d</i> = Detachment/erodibility coefficient (cm ³ /N/s) <i>C%</i> = Clay percentage <i>γ_d</i> = Dry unit weight <i>γ_w</i> = Weight of water (Reference: Innovyze 2013)	Appendix C
C7	$I_{HET} = -\log \left(\rho \frac{10\gamma_w}{\rho} \exp \left[-0.121(C\%)^{0.406} \left(\frac{\rho}{\gamma_w} \right)^{3.10} \right] \right)$	<i>C%</i> = Percentage of clay particles (%passing 0.002mm sieve) <i>ρ</i> = Dry density (kg/cm ³) <i>γ_w</i> = Density of water (0.001kg/cm ³)	Appendix C
D1	$E_r = k_d(\tau_e - \tau_c)^\alpha$	<i>E_r</i> = Rate of erosion (m/s) <i>k_d</i> = Detachment/erodibility coefficient (cm ³ /N.s) <i>τ_e</i> = Effective shear stress (N/m ²) <i>τ_c</i> = Critical shear stress for initiation of erosion (N/m ²) <i>α</i> = Exponent (sometimes assumed as 1)	Appendix D

Appendix B Formulae for determining installed drawdown capacity

B.1 Low-level outlets

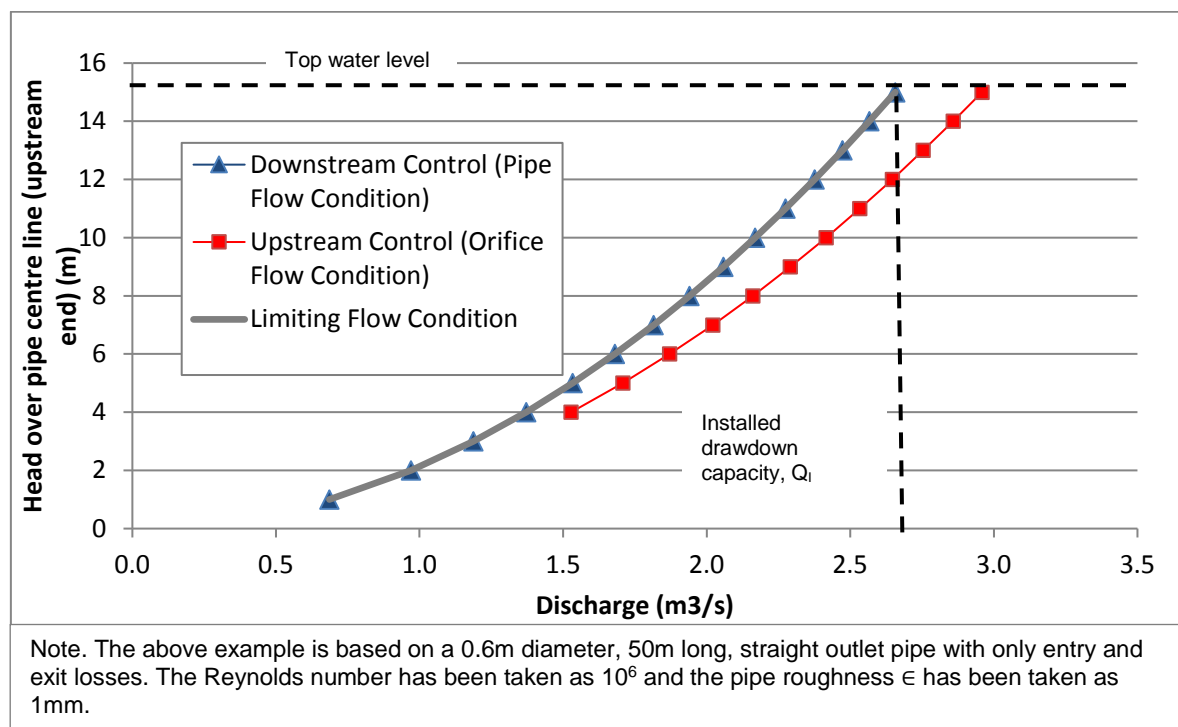
The approach below can be used to estimate the installed drawdown capacity, Q_i , for low-level outlets.

The available hydraulic head is the difference in head between the reservoir surface and the water level at the downstream end of the pipe. Flow through a low-level outlet is generated by this difference in head.

Depending on the specific arrangement and the available head, the rate of flow through the outlet can be governed by either upstream control or downstream control; however, it can be difficult to determine the exact control point without undertaking a detailed analysis. For this reason a simple conservative approach is proposed below to consider both flow conditions separately and take the conservative (minimum) value for the outlet capacity. The two flow conditions and the associated formulae are described below.

The discharge rate should be calculated at varying reservoir levels for both flow conditions (a spreadsheet format lends itself to this) in order to determine the limiting flow condition for each time step. A rating curve can then be plotted for the full range of available heads to determine Q_i , as shown by the example in Figure B.1.

Figure B.1 Example of a rating curve for a low-level outlet



Upstream flow control (orifice flow condition)

To determine the discharge limited by entry into the outlet for a given available head, the orifice equation can be used:

$$Q_{Orifice} = CA\sqrt{2gH} \quad \text{EQUATION B1}$$

Where:

$Q_{Orifice}$ = Discharge limited by entry into the orifice

C = Contraction coefficient (generally around 0.61 – see Chadwick et al. (2013, p.54))

A = Cross-sectional area of the outlet

H = Available hydraulic head, measured from the reservoir water level to the centreline of the orifice

g = Acceleration due to gravity (9.81m/s²)

It should be noted that this equation is only valid for $D < 0.2H$, where D is the outlet diameter.

Downstream flow control (pipe flow condition)

Hydraulic head is a measure of the total energy of the water, and due to the conservation of energy principle the total available head from the reservoir, H , will equal the frictional pipe losses, Δh_f , plus the minor losses, Δh_m . Rearranging the Darcy-Weisbach equation for pipe friction losses⁴ and the minor head loss equation,⁵ yields the following equation which can be used to determine the discharge limited by the pipe flow condition:

$$Q_{Pipe} = A \sqrt{\frac{(2gH)}{\frac{fL}{D} + \Sigma K}} \quad \text{EQUATION B2}$$

Where:

Q_{Pipe} = Discharge limited by pipe flow condition

H = Available hydraulic head, taken as the difference between top water level in the reservoir, and the water level downstream of the outfall pipe

f = Friction factor determined using the Moody chart in Figure B.2.

A = Cross-sectional area of pipe

L = Length of pipe

Darcy-Weisbach equation for pipe friction losses $\Delta h_f = f \frac{L}{D} \frac{V^2}{2g}$

⁵ Minor head loss equation $\Delta h_m = \Sigma K \frac{V^2}{2g}$

$D =$ Pipe diameter

$\Sigma K =$ Sum of local loss coefficients to allow for any 'disruptions' to the flow of water in the pipe, including entries and exits, bends, branches, valves, and contractions and expansions (see Table B.1)

Both equations B1 and B2 assume free outflow at the downstream end of the pipe (i.e. no tailwater influence) and full pipe flow (i.e. $H > D$).

Figure B.2 Moody chart

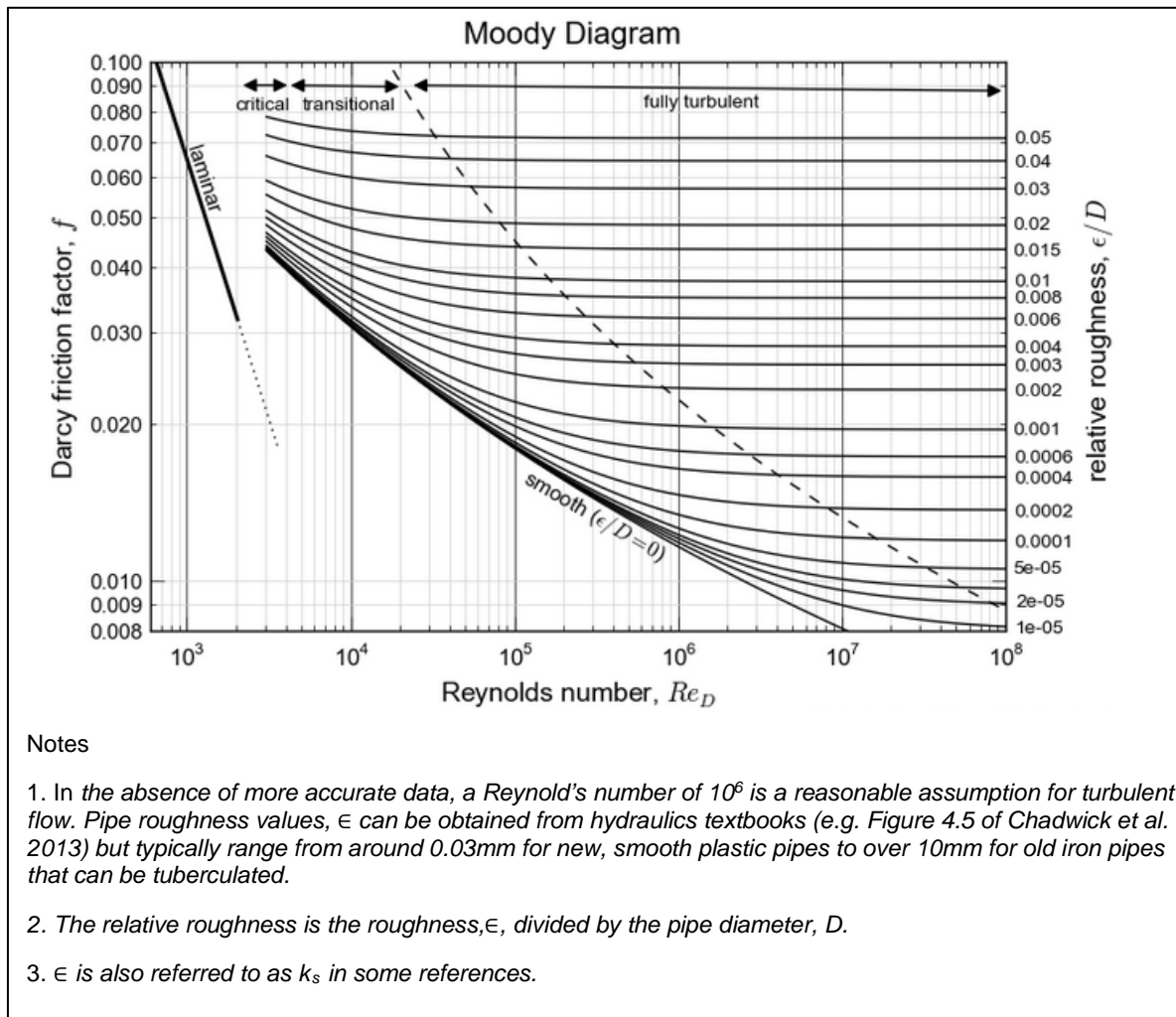


Table B.1 Typical values for local loss coefficients (Miller 1990)

Pipe feature	Minor loss coefficient
Entry	0.5
Exit	1
45° bend	0.11 (for ratio of bend centreline radius to pipe diameter = 1)
90° bend	0.24 (for ratio of bend centreline radius to pipe diameter = 1)
Contraction	$K=(1 - \frac{A_2}{A_1})^2(\frac{A_1}{A_2})^2$
Expansion	$K=(1 - \frac{A_1}{A_2})^2$
Butterfly valve	1 (conservative value for typical commercial valve when fully open)
Diaphragm valve	0.8
Gate valve	0.2

B.2 Siphons

The method of calculating the discharge capacity of a siphon is very similar to that for a low-level outlet described above. The main difference is that due to the length of pipe and associated fittings, flow through a siphon outlet is always governed by the pipe flow condition and the orifice flow condition does not apply. Another important difference is that there is a maximum depth of water a siphon outlet can drain which is limited by the maximum allowable suction head, Z_s . This can be calculated as:

$$Z_s = \frac{P_a}{\gamma} - \frac{P_v}{\gamma} - \frac{V}{2g} - h_{fp} \quad \text{EQUATION B3}$$

Where:

$$\frac{P_a}{\gamma} = \text{Atmospheric head which is 10m}$$

$$\frac{P_v}{\gamma} = \text{Vapour pressure head which is 0.2m at 15°C}$$

$$\frac{V^2}{2g} = \text{Velocity head in the suction pipe}$$

$$h_{fp} = \text{Friction loss in the suction pipe}$$

As a rule of thumb Z_s cannot exceed around 5 to 6m to avoid potential cavitation, flow separation and ultimately siphon break.

Siphon design and analysis is a specialist subject and should be carried out by appropriately experienced and qualified hydraulic engineers.

B.3 Pumps

The stated hydraulic capacity of pumps from pump suppliers should be treated with caution as trial drawdown tests have suggested significantly lower capacities may be achieved in actual installations. Typical pump capacities and mobilisation times found during such trials are given in Table B.2.

Table B.2 Typical pump capacities and mobilisation time found during trials

Pump size	Maximum flow rate (l/s)	Typical mobilisation time in an emergency from hire depot (excludes journey time)
6 inch	55	Around 4 hours
8 inch	95	
10 inch	250	Not commonly in stock therefore around 2 days

Appendix C Parameters required to estimate time to failure

As discussed in Section 6.4.2, in order to assess the overall vulnerability of a dam to rapid failure the following two parameters need to be determined:

- the hydraulic gradient through the dam, i
- the erodibility of the dam fill (i.e. the erosion rate index, I_{HET})

Guidance on determining these parameters is given below.

C.1 Calculation of hydraulic gradient

The hydraulic gradient across the dam governs the rate at which internal erosion will occur and thus the speed at which a reservoir might need to be drawn down. Where the critical failure location is at the base of the dam fill, the hydraulic gradient should be calculated using Equation C1.

$$\text{Hydraulic gradient, } i = \frac{H}{L} \quad \text{EQUATION C1}$$

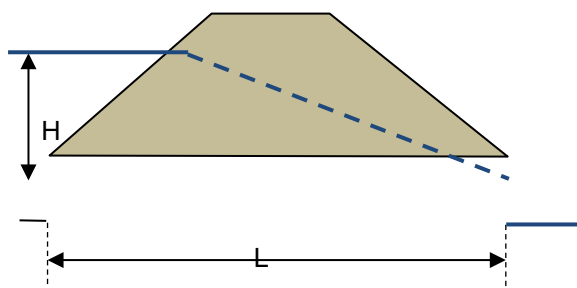
Where: H = Maximum retained depth of water (m)

L = Effective seepage length, i.e. the maximum horizontal width of the water-retaining element of the embankment at the location of H (m)

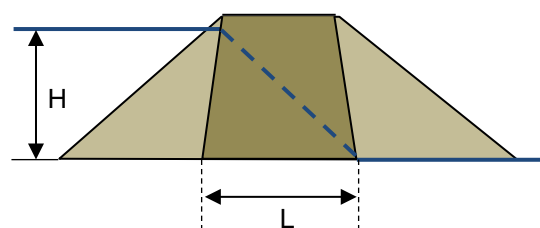
The measurement of L will depend on any zoning of the fill within the embankment cross-section. In most cases where the embankment is homogeneous or where the shoulders comprise a generally cohesive material then L should simply be taken as the base width of the embankment (see Figure C.1a). In the case of a clay core dam with permeable shoulders (e.g. granular shoulders with a permeability exceeding around $1 \times 10^{-5} \text{m/s}$) the length, L , should be taken as the base width of the core only (see Figure C.1b).

Figure C.1 Determining hydraulic gradient

a) Homogeneous dam or dam with generally cohesive shoulders



b) Clay core dam with permeable shoulders



The above approach will give the hydraulic gradient through the base of the dam which for a typical embankment, such as that illustrated, is the location where it is greatest. However, this will not always be an appropriate level and for the potential failure scenario it may be necessary to consider the hydraulic gradient at a different location. Instances when the approach may need to be modified include:

- where the dam foundation is more erodible than the dam fill
- where there are changes in geometry such that the risk of internal erosion is greater further up the body of the dam (e.g. if there is a berm, the hydraulic gradient may be greatest just above the berm)
- where there are changes in internal grading such that the risk of internal erosion is greater further up (e.g. zones of higher erodibility fill)
- where there are more likely seepage paths elsewhere (e.g. along the outside of conduits, decaying tree roots)

The approach above is simplified for ease of assessment; however it is acknowledged that for non-homogeneous dams this may lead to overestimating the theoretical time it would take a dam to fail by internal erosion. In this case it may be necessary to consider the hydraulic gradient and the safeguards against internal erosion (e.g. filters) in more detail. For example flow limitation may occur where there is a relatively fine-grained granular material upstream of the core. As noted in Section 8.2.3 of ICOLD (2013) the zones upstream and downstream of the core can generate high hydraulic losses and this may be taken into account for a more detailed assessment.

Similar considerations apply to other types of dams (e.g. where there is a watertight membrane or concrete face slab on the upstream face, or concrete/masonry walls within the cross-section). In these cases, an assessment should be made to consider how these features would limit any seepage flow through the embankment and whether or not the maximum flow would generate the critical shear stress for the dam fill material (see Appendix D.2).

C.2 Standard method to estimate the erosion rate index

The erodibility of the dam fill is a key parameter in assessing the vulnerability of a dam to rapid failure. The most common parameter used to define soil erodibility is the erosion rate index, I . The higher the index the longer it takes for a soil to erode. There are several test methods which can be used to measure erosion rate index and the test method is notated in subscript to the term I ; for example, the most widely used test is the hole erosion test (HET) hence the parameter is often termed I_{HET} .

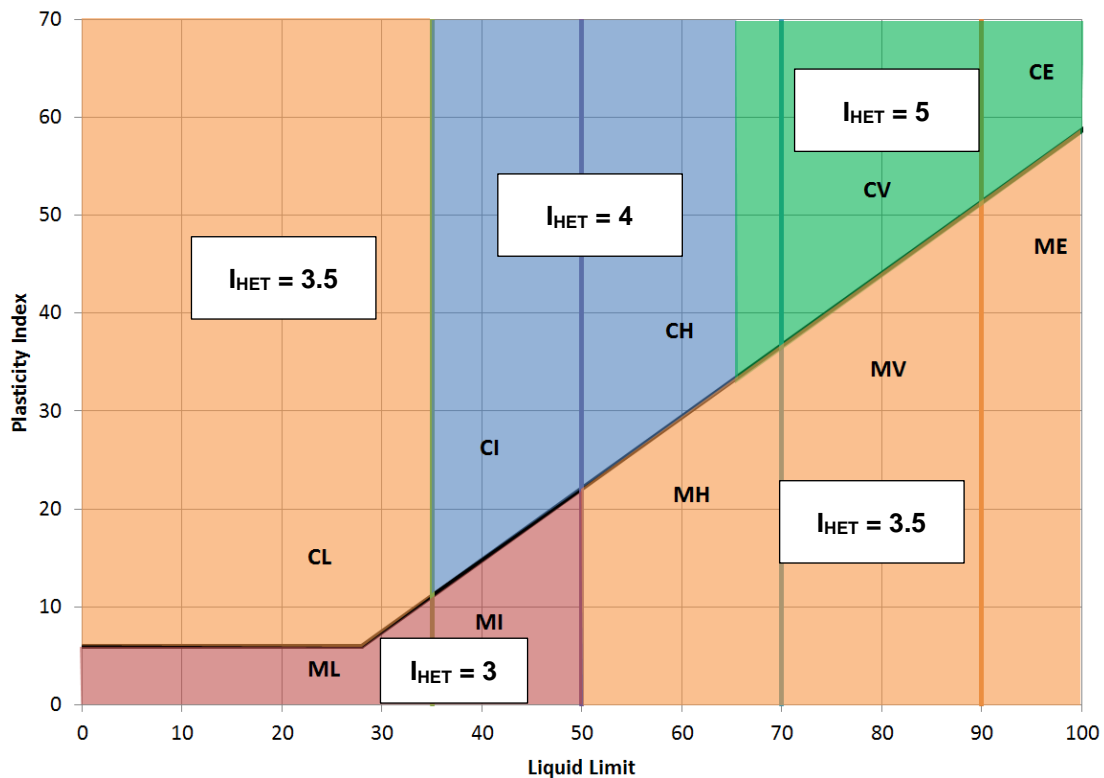
Erosion rate index values are rarely available for existing dams and a correlation with more commonly known soil parameters can be used to evaluate appropriate index values.

The simplest way to estimate the erosion rate index is to use the correlation with soil particle size and plasticity given in Table 3.4 of ICOLD (2013). The correlation is reproduced in Table C.1 and plotted on a standard A-Line plot in Figure C.2. The potential inaccuracy of this method is indicated by the wide range of values in Table C.1 between the 'likely minimum' and 'likely maximum'. For an initial assessment, the 'best estimate' values should be adopted unless there is evidence for selecting a lower value.

Table C.1 Erosion rate index versus soil classification (reproduction of Table 3.4 from 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

Figure C.2 Erosion rate index, I_{HET} , for fine-grained soils based on soil classification



Other methods to determine the erosion rate index, including laboratory test methods and other correlation-based methods, are described in Appendix C.3.

For dams with a central core (even if the shoulders are also cohesive) a conservative approach would be to take the erosion rate index for the core material, rather than taking an overall average for the core and shoulders. If the critical location for internal erosion is considered to be through the foundation (see Section 2.7) the erosion rate index for the foundation should be considered.

C.3 Alternative methods to determine erosion rate index

C.3.1 Overview of alternative methods

Appendix C.2 describes a simple method to estimate the erosion rate index based on correlation with soil classification. Table C.2 summarises various alternative methods which may also be used, in approximate order of accuracy. Because the time to failure is sensitive to this parameter, the more accurate methods are recommended to justify the selection of higher values of I_{HET} ; however, the rapid assessment described in Appendix C.2 is likely to be sufficient in many cases.

Table C.2 Methods for determining erosion rate index

	Method of determining erosion rate index	Examples of when method would be appropriate	Reference
Most accurate	Laboratory testing		
	Hole erosion test (HET) (Note 1).	Provides a more accurate value of I_{HET} where the rate of erosion is felt to be critical.	Appendix C.3.2
	Equations based on statistical correlation of laboratory tests		
	Equations published by Wan et al. (2002), which are based on statistical correlation of results from HET tests for a variety of soil types. Two equations are given, one for coarse-grained soils and one for fine-grained soils. The equations require a number of geotechnical parameters to be known (Note 2).	Where detailed geotechnical data exists to enable the equations to be used.	Appendix C.3.3
Least accurate	Rapid assessment linked to soil plasticity		
	Published correlation from ICOLD (2013) linked to basic soil classification index tests. This correlation is reproduced in Table C.1 and shown graphically in Figure C.2.	Likely to be used in most cases, at least as an initial assessment.	Appendix C.2
	Equation based on the detachment coefficient, K_d		
	This is a simpler but more approximate equation based on the percentage clay and dry density of the soil. The equation was first published by Temple and Hanson (1994) but is also referenced widely in other literature.	May be useful as an additional check for one of the other methods.	Appendix C.3.4
Notes:			
1. Other types of laboratory test are also possible as detailed below.			
2. The equations given by Wan et al. (2002) for fine-grained soils requires a pinhole test classification for the soil. A pinhole test may be a more readily available test than the HET but where possible an HET test is strongly recommended.			
3. The last two methods are probably comparable in terms of level of accuracy.			

C.3.2 Laboratory testing

The most accurate method of determining erosion rate index is to measure it directly in a laboratory on samples of soil. There are several test methods currently available as described in Table C.3. Further tests, including those used to assess dispersivity are described in Section 7.8.3.6 of the *International levee handbook* (CIRIA 2013).

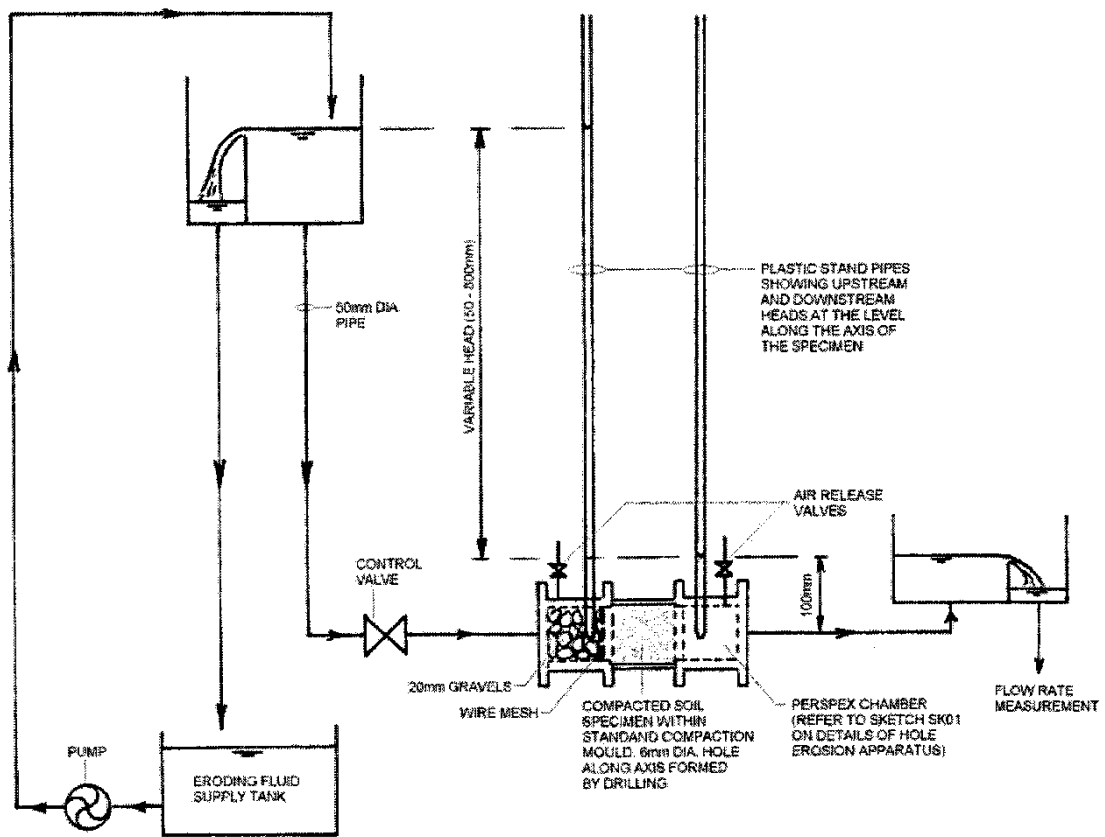
Table C.3 Physical test methods

Test	Description	Key references
Hole erosion test (HET)	Developed to be faster and more economical than the SET (see below). Utilises an internal flow through a hole pre-drilled in a soil specimen, to represent the flow condition that occurs during piping erosion of embankment dams. Utilises a standard mould from the Standard Compaction Test and a 6mm drilled hole. Flow rate is used as an indirect measurement of hole diameter (see Figure C.3).	Wan and Fell (2002 and 2004)
Jet erosion test (JET)	Utilises a submerged jet to produce scouring surface erosion, similar to that which occurs at a headcut or free overflow.	Hanson and Cook (2004)
Slot erosion test (SET)	A 2.2mm wide by 10mm deep by 1m long slot is artificially formed along one side of an unsaturated soil sample in a rigid sample box and covered with a layer of clear Perspex. The widening of the slot is observed through the Perspex as flow is passed down the slot.	Wan and Fell (2002 and 2004)
Rotating cylinder test	Erosion measured from vertical sides of a 100mm diameter by 100mm high sample.	Lim and Khalili (2010)
Note 1. See also Table 7.113 of the <i>Internal levee handbook</i> (CIRIA 2013).		

The most widely used tests at this time are the hole erosion test (HET) and the jet erosion test (JET). Both tests are fully described in the literature and utilise relatively straight forward test equipment that can be readily reproduced in most soil testing laboratories.

ICOLD (2013) notes that the HET and JET methods can give different estimates of erosion rate index. This was also found by Wahl et al. (2009) who compared the HET and JET and concluded the HET generally indicates slower rates of erosion (higher I values). Because much of the literature is authored by the developers of either test it is difficult to make an impartial recommendation for which test is better. However, it is noted that the HET set-up better replicates the mechanism of internal erosion. Two of the other more rapid methods shown in Table C.2 are also linked to HET results so, for these reasons, this guide is based on erosion rate index as measured by the HET, I_{HET} .

Figure C.3 Schematic diagram of a hole erosion test assembly (Wan et al. 2002)



C.3.3 Equations based on statistical correlation of laboratory tests

Wan et al. (2002) carried out statistical analysis of test data from the hole erosion test for a variety of different soil types and developed two equations correlating the erosion rate index with other soil parameters. The equations are reproduced below with Equation C2 applying to coarse-grained soils and Equation C3 applying to fine-grained soils.

For coarse-grained soils:

$$I_{HET} = 6.623 - 0.016\rho_d - 0.104 \frac{\rho_d}{\rho_{dmax}} - 0.044\omega - 0.074\Delta\omega_r + 0.113S + 0.061Clay(US)$$

EQUATION C2

Where: I_{HET} = Erosion rate index

ρ_d = Dry density of the soil (Mg/m^3)

$\frac{\rho_d}{\rho_{dmax}}$ = Percentage compaction (%)

ω = Water content (%)

$\Delta\omega_r$ = Water content ratio relative to optimum moisture content,
i.e. $\Delta\omega_r = \frac{(\omega - OMC)}{OMC}$ (%)

S = Degree of saturation (%)

$Clay(US)$ = Mass fraction finer than 0.005mm (%)

For fine-grained soils:

$$I_{HET} = -10.201 + 9.572\rho_d - 0.042 \frac{\rho_d}{\rho_{dmax}} + 0.103\omega + 0.0097\Delta\omega_r - 0.0056Fines + 0.042Clay(US) - 0.090LL + 0.111I_P + 0.443Pinhole$$

EQUATION C3

Where: *Fines* = Fines content (<0.075mm) (%)

LL = Liquid limit (%)

I_P = Plasticity index (%)

Pinhole = Pinhole Test Classification expressed as an ordinal number, i.e. '1' for Class D1, '2' for Class D2, '3' for Class PD1, ..., '6' for Class ND1

As noted in Note 2 of Table C.2, if pinhole test data does not already exist then it is preferable to carry out a hole erosion test directly rather than carry out pinhole tests for use in this equation.

C.3.4 Equation based on the detachment coefficient, k_d

Temple and Hanson (1994) investigated headcut development in earth spillways and carried out regression analysis of the soil parameters common between the studies to develop an equation for erosion rate, which they termed the detachment coefficient, k_d ($\text{cm}^3/\text{N/s}$). This coefficient is referred to in various texts and is used for example in many software models for dam breach analysis.

The detachment coefficient is related to the erosion rate index, I_{HET} , as shown in equations C4 to C6.

$$I_{HET} = -\log C_e \quad \text{EQUATION C4}$$

Where: C_e = Detachment/erodibility coefficient

(References: Hanson et al. 2010, ICOLD 2013)

$$C_e = k_d/\rho \quad \text{EQUATION C5}$$

Where: k_d = Detachment/erodibility coefficient ($\text{cm}^3/\text{N/s}$)
 ρ = Dry density

(Reference: Hanson et al. 2010)

$$k_d = \frac{10\gamma_w}{\gamma_d} \exp \left[-0.121(C\%)^{0.406} \left(\frac{\gamma_d}{\gamma_w} \right)^{3.10} \right] \quad \text{EQUATION C6}$$

Where k_d = Detachment/erodibility coefficient ($\text{cm}^3/\text{N/s}$)
 $C\%$ = Clay percentage
 γ_d = Dry unit weight
 γ_w = Weight of water

(Reference: Innovyze 2013)

These equations can be rearranged to express I_{HET} in terms of the percentage of clay particles within the soil and its dry density as follows:

$$I_{HET} = -\log\left(\rho \frac{10\gamma_w}{\rho} \exp\left[-0.121(C\%)^{0.406} \left(\frac{\rho}{\gamma_w}\right)^{3.10}\right]\right) \quad \text{EQUATION C7}$$

Where: $C\%$ = Percentage of clay particles (% passing 0.002mm sieve)
 ρ = Dry density (kg/cm³)
 γ_w = Density of water (0.001kg/cm³)

Erosion rate indices obtained using this equation have been compared against the representative values in Table C.1 and test results published in Wahl et al. (2009). Overall the equation appears to often give lower values (i.e. indicates higher erodibility) than the correlation with soil classification.

Appendix D Assessing the vulnerability to rapid failure by internal erosion

D.1 Time to failure

Two key factors govern the time it would take internal erosion to progress to failure and these need to be understood in order to assess the vulnerability of a dam to rapid failure:

- the hydraulic gradient through the dam, i
- the erodibility of the dam fill (i.e. the erosion rate index, I_{HET})

Guidance on determining these parameters is given in Appendix C.

Figure D.1 shows a simplified approach which can be used to obtain a theoretical drawdown rate to avert internal erosion failure, D_0 , based on these two parameters. This is the approximate estimated drawdown rate that would be required to draw a reservoir down in the time it would otherwise take for catastrophic failure to occur. The derivation of the graph is summarised in Information box D.1. The graph is based on a number of simplifications and assumptions and is only intended to give an approximate indication of relative drawdown rates. It is emphasised that the values of D_0 derived from the graph are highly sensitive both to the basis for the assumed hydraulic gradient, and to the value adopted for the erosion rate index, both of which are based on parameters that are often uncertain. The assessment should therefore be made by experienced engineers exercising appropriate judgement.

Information box D.1 – Key principles of how Figure D.1 was derived

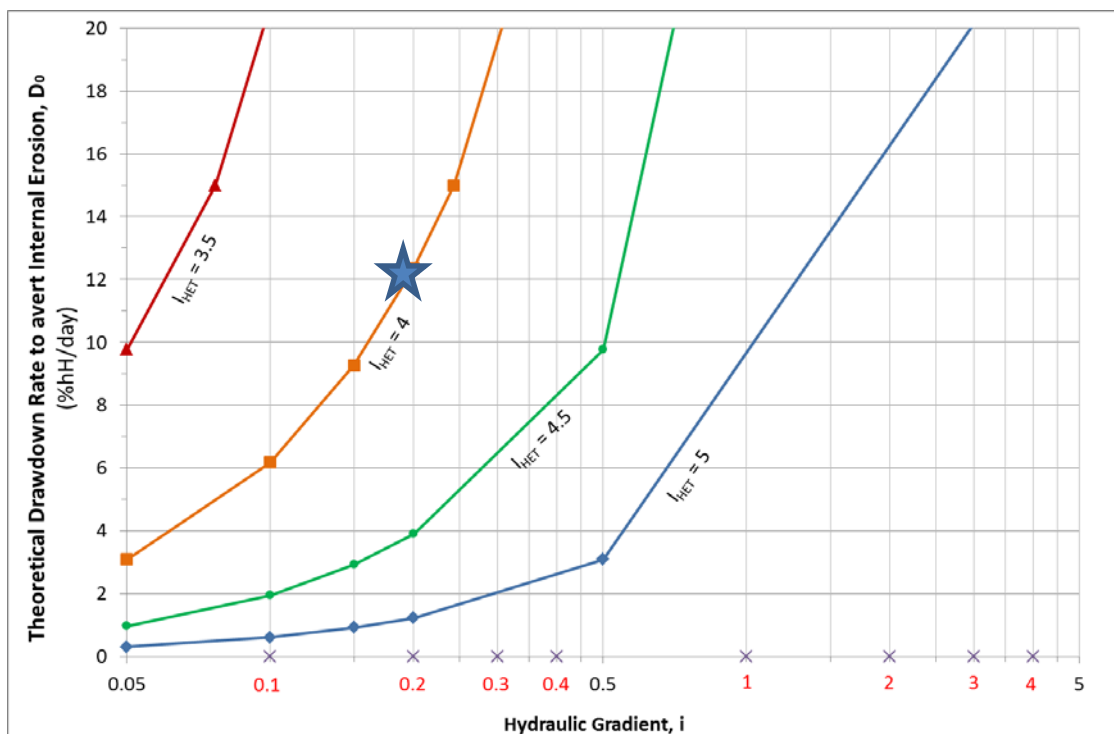
- The graph is based on the predicted likely rate of internal erosion, using a relationship similar to that published in Figure 8.1 of ICOLD Bulletin 164 (ICOLD 2013). This relationship links 'time to failure' to two key variables:
 - the hydraulic gradient through the dam, i
 - the erodibility of the dam fill (i.e. the erosion rate index, I_{HET})
- The original relationship published by ICOLD defined the 'time to failure' as the time it would take for a 25mm diameter hole to widen to 1m diameter. However, it is assumed a leak would be detected before the hole reached this size such that drawdown to avert failure could be commenced earlier at a slower rate. Therefore Figure D.1 is instead based on the 'time to failure' starting from an initial hole size of 5mm which represents the point at which a concentrated leak may first be detectable. Because of the exponential rate of increase in internal erosion with hole size this adjustment has a relatively significant effect on the drawdown capacity.
- A simplified approach was developed using the adjusted relationship to analyse iteratively the remaining time to failure for a particular dam, at small time-steps during a drawdown scenario (i.e. as time progresses the eroded hole size increases but the water level falls and thus hydraulic gradient decreases).
- A trial and error approach was adopted using the model to determine the minimum initial drawdown rate required such that the hole size never quite reached 1m diameter.
- The analysis was repeated for a range of input parameters representative of UK reservoirs and the results are plotted in Figure D.2. Sensitivity studies showed that the results were relatively insensitive to the height–storage relationship of the reservoir.

- vi. Several simplifications and approximations were made in order to produce the graph, including:
- In keeping with the relationship published in ICOLD Bulletin 164, Figure D.1 conservatively takes no account of critical shear stress. This means that the theoretical drawdown rates may be **overestimated**. Critical shear stress should be considered separately as explained in Appendix D.2.
 - Due to the complexity of modelling the actual hole size at each step, the current time to failure at each time step, has been taken as the time for a hole to develop from 5mm to 1,000mm (i.e. using the relationship discussed in point ii above). 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 rates in Figure D.1 are deemed appropriate for gaining a rough indication of the theoretical rate required but, as with any theoretical models of this type, the results should be considered within an overall framework of engineering judgement.

Because of conservatism in the analysis (e.g. the critical shear stress is assumed to be zero as discussed in Information box D.1), the time required for detecting the failure mode and activating drawdown was not explicitly included.

Figure D.1 Theoretical drawdown rate to avert internal erosion, D_0



The star shows the assumed typical dam referred to in Table 6.2 (the drawdown rates recommended in Table 6.2 are lower than the value of D_0 indicated above because of the critical shear stress as discussed in Appendix D.2)

The values of D_0 given in Figure D.1 are based on a reference reservoir shape where the valley is approximately V-shaped and the storage capacity increases exponentially with height. The rates may need to be adjusted for other shapes of reservoir basin.

Figure D.1 conservatively takes no account of critical shear stress. This concept is discussed in Appendix D.2 and means that in practice some dams with lower erodibility and hydraulic gradient would probably not actually progress to failure.

It is noted that research and understanding of internal erosion rates in dams is still in its infancy and further improvements in analytical techniques are likely in future years. The approach adopted here aligns with current best practice but is only intended to give a broad indication of the relative theoretical drawdown rates that may be required. Further information on the current maturity of internal erosion knowledge can be found in the references listed in Section 6.4.1.

D.2 Critical shear stress

The erosion rate of soil is often expressed using the excess stress equation (e.g. Hanson et al. 2010) which states that:

$$E_r = k_d(\tau_e - \tau_c)^\alpha \quad \text{EQUATION D1}$$

E_r = Rate of erosion (m/s)

k_d = Detachment/erodibility coefficient (cm³/N.s)

τ_e = Effective shear stress (N/m²)

τ_c = Critical shear stress for initiation of erosion (N/m²)

α = Exponent (sometimes assumed as 1)

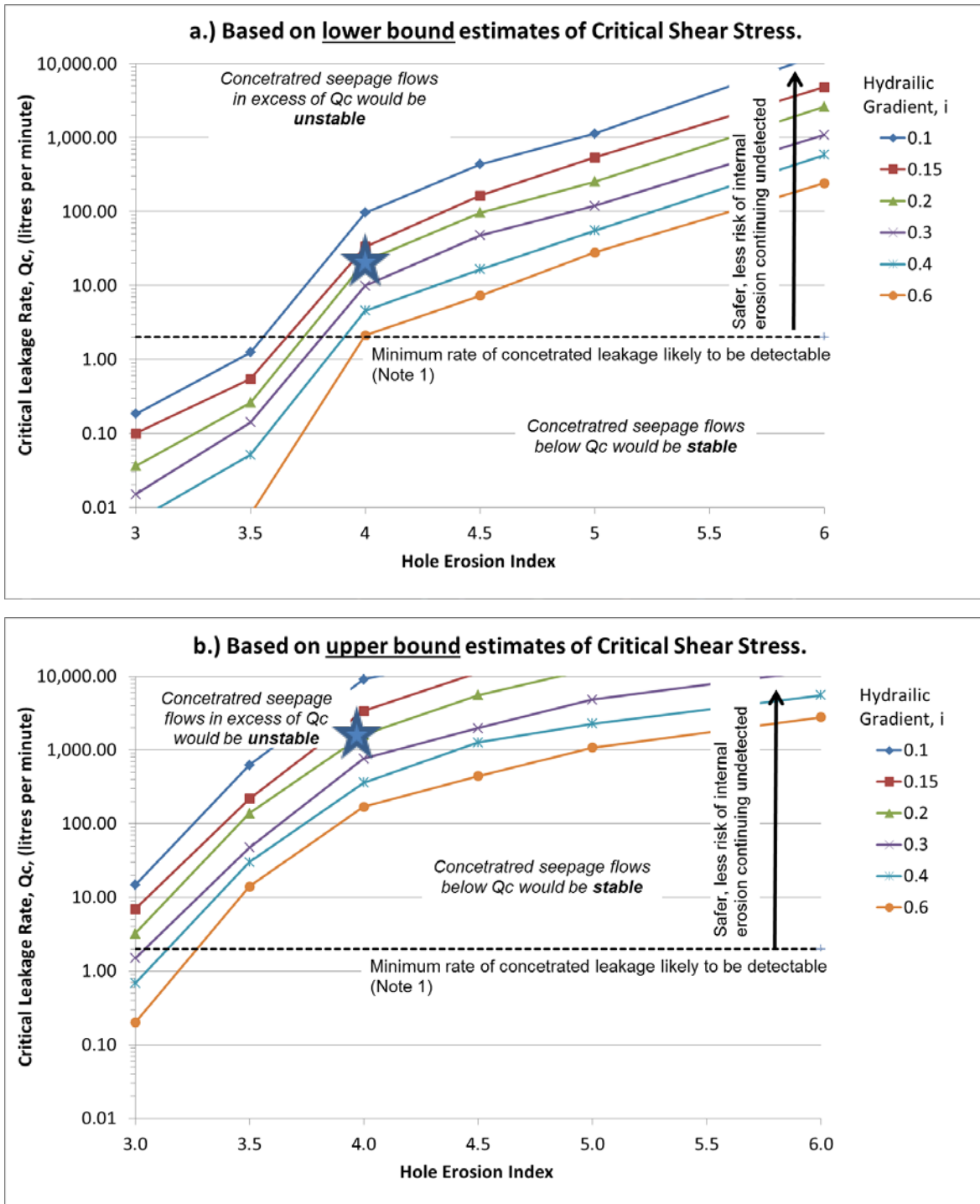
This suggests that erosion would only initiate once the shear stress generated by the flowing water exceeds a certain threshold known as the critical shear stress. Table 3.5 of ICOLD Bulletin 164 (ICOLD 2013) gives approximate estimates and the likely range of critical shear stresses for different soil types.

The reference position assumed in deriving the values of D_o in Appendix D.1 is based on the critical shear stress being zero. This is a conservative assumption, particularly for higher values of I_{HET} , but is consistent with the approach taken in Figure 8.1 of ICOLD Bulletin 164.

Figure D.2 shows how the critical leakage rate Q_c (i.e. the flow rate at which the critical shear stress is likely to be exceeded) depends on the hydraulic gradient across the dam and the erodibility of the fill. The two graphs show the relationship for upper and lower bound estimates of the critical shear stress respectively (based on the likely range taken from Table 3.5 of ICOLD 2013). Internal erosion would not progress unless the critical leakage rate is exceeded. Therefore where the graphs show the critical leakage rate would be relatively high then it is likely that leakage would remain steady and could be detected and dealt with. However, where the critical leakage rate is relatively small there is a risk that by the time it is detectable, internal erosion could be well progressed and the size of the leakage path and the amount of leakage could be propagating rapidly towards failure.

The graphs in Figure D.2 are based on a number of simplifying assumptions. For example they assume a cylindrical leakage hole with no imperfections which would cause localised variations in shear stress.

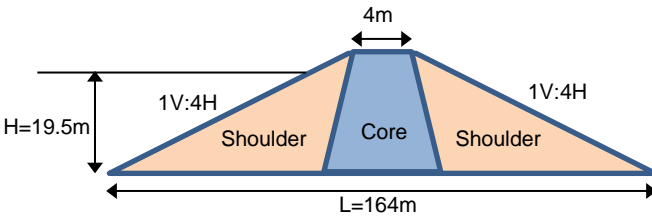
Figure D.2 Critical leakage rates, Q_c , at which internal erosion will continue



Notes.

1. The minimum rate of concentrated leakage which is likely to be detectable has been suggested as 2 litres/minute based on expert elicitation (Brown and Aspinall 2004). Dispersed seepage and wet spots may be detectable at lower flow rates.
2. Published values of critical shear stress (ICOLD 2013) are not differentiated for low values of I_{HET} below 3.
3. The above graphs assume non-dispersive soil. The critical shear stress (and thus critical leakage rates) for dispersive soils are significantly lower (see Table 3.5 of ICOLD Bulletin 164).
4. ★ The star shows the assumed typical dam referred to in Table 6.2.

Appendix E Worked example

Consideration	See guide section	Evaluation	Conclusion
Site characterisation			
Installed drawdown facilities	3.2	The installed drawdown capacity is provided by a single low-level outlet with the valves operated from a headstock at the upper level of the upstream valve tower.	
Reservoir inflows	3.3	Catchment area = 8km ² (no indirect catchment). Reservoir surface area, A = 430,000m ² (43ha) Reservoir volume = 2,800,000m ³ There is a gauging station at the upstream end of the reservoir which can be used to produce a flow duration curve (daily flows). The flows have been adjusted based on the relative catchment areas of the reservoir and the gauge site. There is no indirect catchment.	A = 430,000m² Q₅₀ = 0.4m³/s Q₁₀ = 1.6m³/s
Ability to divert inflows	3.4	No by-wash channel or any other means to store or divert inflows	Q_x = 0m³/s
Dam geometry, construction and foundation	Appendix C.1 Appendix C.2	 <p>Core: intermediate plasticity clay Shoulders: low plasticity clay Hydraulic gradient, i, where shoulders are low permeability $= H \div L = 19.5 \div 164 = 0.1189$ No reliable testing data is available for this dam relating to erosion rate index, therefore use rapid assessment method.</p>	H = 19.5m L = 164m i = 0.12 I_{HET} = 4

		<p>Foundation: relatively non-erodible compared to dam fill There are unconfirmed reports that the dam may have a sand chimney filter but there are no design records.</p>	
Failure modes	3.5	Critical failure mode is considered to be internal erosion. The dam was raised by 3m in the past so this interface is a potentially vulnerable location for internal erosion.	
Consequences of failure	3.6	LLoL = 100 Cost of third party property damage ~£10 million	Dam category: A
Constraints on drawdown capacity	3.7	Structural reliability Valves 20 years old. They have been tested within the last year and operated without problems. Operational reliability Staff on call 24/7 but site remote with poor mobile reception, but reasonable access road. Bridge access required to operate valves and in good structural condition	Drawdown capacity is considered reliable.
Surveillance and activation processes	3.8	Site visual inspection twice a week, therefore detection of defects would average 42 hours after becoming visible.	
Determining installed drawdown rate			
Installed discharge capacity, Q_I	4.1	Capacity verified by testing 5 years ago.	$Q_I = 3.0\text{m}^3/\text{s}$
Reliable discharge capacity, Q_R	4.2 to 4.4	Drawdown capacity is considered reliable so $Q_R = Q_I$	$Q_R = 3.0\text{m}^3/\text{s}$
Inflow pass-through allowance, Q_P	5.2	$Q_P = Q_{50} - Q_X = 0.4 - 0 = 0.4$	$Q_P = 0.4\text{m}^3/\text{s}$

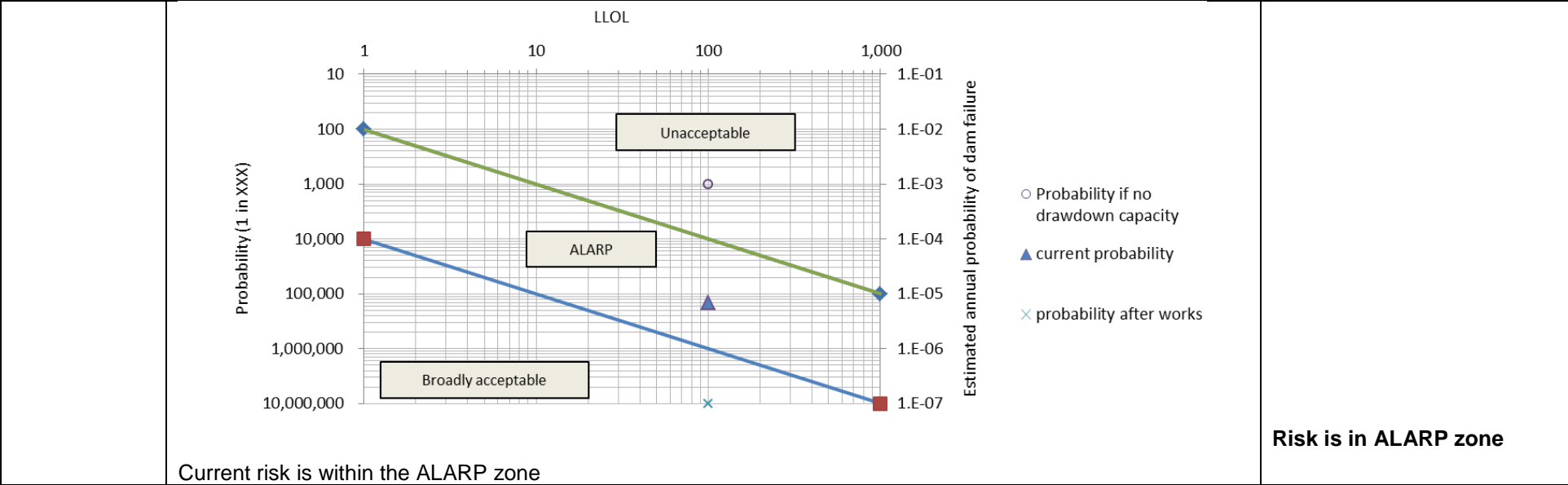
		(If sensitivity check is required:- $Q_{10} - Q_X = 1.6 - 0 = 1.6\text{m}^3/\text{s}$)	$(Q_{10} \text{ sensitivity check} = 1.6\text{m}^3/\text{s})$
Discharge capacity available for reservoir lowering, Q_L	5.1	$Q_L = Q_R - Q_P = 3.0 - 0.4 = 2.6$	$Q_L = 2.6\text{m}^3/\text{s}$
Installed drawdown rate, D_I	5.3.1	$D_I = 100 \times (Q_L \times 86,400) \div (A \times H) =$ $100 \times (2.6 \times 86,400) \div (430,000 \times 19.5) = 2.7$	$D_I = 2.7\%H/\text{day}$
		Conversion of D_I to mm/day: $(1,000 \times D_I \times H) \div 100 = (1,000 \times 2.7 \times 19.5) \div 100 = 523\text{mm}/\text{day}$	(D_I equates to 523mm/day)
Time to empty the top third of the reservoir volume, $T_{33\%}$	5.3.2	The top 6.5m (33%H) of the reservoir contains roughly 50% of volume. Step method used to estimate $T_{33\%}$.	$T_{33\%} = 7 \text{ days}$ (14 days if inflows were Q_{10})
Assessing adequacy of installed drawdown rate			
Consideration 1: Basic recommended standard	Basic recommended standard	6.3	D_I is less than the recommended minimum rate for a Category A dam (5%H/day ref. Table 6.2). The upper cap of 1m/day does not apply.
	Overview	Installed capacity does not meet basic recommended standards.	
Consideration 2: Vulnerability to rapid dam failure	Overall vulnerability	6.4.2	<p>Subject dam is less vulnerable to rapid failure than reference dam assumed for basic standard in Table 6.2. The required theoretical drawdown rate of 7%H/day is 60% of the theoretical drawdown rate for the reference dam.</p>
			Theoretical drawdown rate required is 60% of the theoretical drawdown rate for the reference dam

	Conditions that may inhibit internal erosion initiating	6.4.3	Dam material is cohesive. There are no design records showing the possible sand filter.	Inhibiting features cannot be confirmed as reliable
	Conditions that may inhibit internal erosion continuing	6.4.4	None known.	None
	Overview	No strong case for adjusting basic recommended standard at this point, but some reduction could be allowed		
Consideration 3: Other factors	Frequency and quality of surveillance	6.5.1	Surveillance frequency is good to very good.	Similar to reference dam
	Time required to activate drawdown	6.5.2	Activation requires operative to attend site with padlock key and authorisation from reservoir manager. Assume this could take up to 2–3 hours.	Similar to reference dam
	Alternative emergency actions	6.5.3	Drawdown is the only action covered by the on-site plan. No alternative emergency actions are considered practical.	No effect on the required drawdown capacity
	Time to empty the upper third of the reservoir depth	6.5.4	T _{33%} has been calculated to be 7 days (14 days if inflows were Q ₁₀). The most vulnerable zone for internal erosion is above this depth at around 3m below the crest. Drawdown to this level would take 4 days (8 days with Q ₁₀ inflows). On the basis that the dam is not particularly susceptible to internal erosion this is considered acceptable. Figure D.2 indicates that concentrated leakage flows would need to exceed at least ~50 litres/minute before internal erosion continued. It is considered unlikely that such leakage rates would develop in 4 days (or even 8 days).	The time to empty a significant portion of the reservoir is deemed acceptable
	Ability to pass flood flows while repairs are implemented	6.5.5	There is sufficient reservoir volume in the lower part of the reservoir to attenuate foreseeable floods during the likely period of repairs, if the outlet was opened.	No additional drawdown capacity is required for this purpose
	Overview	No strong case for adjusting basic recommended standard		
Consideration 4: Precedent practice	Formula for target capacity	6.6.1	$300 + (5 \times H) + (8,640 \times Q_{10} \div (A \div 10,000)) =$ $300 + (5 \times 19.5) + (8,640 \times 1.6 \div (430,000 \div 10,000)) =$ 719mm/day Installed capacity (D ₁) = 523mm/day	Installed drawdown rate <u>does not meet</u> this precedent
	1m/day rule	6.6.2	1m/day = 1,000mm/day > 523mm/day	Installed drawdown rate <u>does not meet</u> this precedent
	Canal & River Trust approach	6.6.3	Consequence class is A1 (100 lives at risk) and surveillance is twice a week, therefore 50% volume to be removed in 5 days. 50% of the volume is within the top third of the reservoir depth and	Installed drawdown rate <u>does not meet</u> this precedent

			could be emptied within 14 days under Q ₁₀ inflows (which are similar to winter daily mean inflow).	
	USBR standard	6.6.4	Taking conservatively the most onerous evacuation rate (high hazard, high risk) requires level to be reduced to 75%H in 10 to 20 days. Installed facilities achieve a reduction to 75%H in 6 days under Q ₅₀ inflows and 11 days under Q ₁₀ inflow.	Installed drawdown rate meets precedent
	Overview	Installed drawdown rate only satisfies one of the four precedent practices considered		
Overall evaluation		Installed drawdown rate is not adequate		

Mitigation for insufficient drawdown capacity

Guide section	Assessment	Conclusion
Identify feasible mitigation options		
8.2 and Table 8.1	Option 1. Improve/refurbish existing facilities Install a penstock in the spillway weir crest to increase the drawdown rate (the sustained drawdown rate was less of an issue). Estimated overall project cost would be £45,000.	Scheme cost = £45,000
Form of assessment to be adopted		
8.3.1	Quantitative approach is appropriate.	QRA
Quantitative risk assessment		
8.3.3 (Table 8.2)	<p>Quantify the reduction in probability of failure</p> <p>Assign index probabilities of failure based on Option A from Table 8.2:</p> <ul style="list-style-type: none"> • Index probability with no drawdown capacity: 1×10^{-3} (1 in 1,000) • Index probability with full recommended drawdown capacity: 1×10^{-7} (1 in 10,000,000) <p>Capacity as % of recommended drawdown capacity is $100 \times 2.7/5 = 54\%$ \therefore interpolated probability of failure with current capacity (log-linear relationship) $= 10^{(\text{Log } 1\text{E-}3 - 0.54 \times (\text{log } 1\text{E-}3 - \text{log } 1\text{E-}7))} = 6.9 \times 10^{-6}$</p> <p>Plot current probability on an FN-Chart (e.g. Figure 9.2 of RARS or similar). LLoL is 100 lives.</p>	



	<p>Assess if cost of works satisfies ALARP</p> <p>Reduction in annual probability of failure = Current probability – Probability after works = $6.9E^{-6} - 1.0E^{-7} = 6.8E^{-6}$</p> <p>Present value (PV) of likely savings in lives = (Reduction in probability x LLoL) x PV discount = $(6.8E^{-6} \times 100) \times 30 = 0.02$ lives</p> <p>PV of savings for third party damage = Reduction in probability x Cost of third party damage x PV discount = $(6.8E^{-6} \times \text{£}10 \text{ million}) \times 30 = \text{£}2,000$</p> <p>Cost to prevent a fatality (CPF) = $\frac{\left(\text{Cost of refurbishing existing facilities} \right) - \left(\text{PV risk savings for third party damage} \right)}{\text{PV savings in lives}}$</p> <p>$= \frac{\text{£}45k - \text{£}2k}{0.02} = \text{£}2.3 \text{ million}$</p> <p>Value of preventing a statistical fatality (VPF) = £2 million Assume proportion factor (PF) is 5 since the cost estimates are accurate and the probability of failure is low. Therefore CPF would need to be 5 x £2 million = £10 million before refurbishment of existing facilities would be considered disproportionate. CPF < £10 million, therefore refurbishment of existing facilities is justified.</p>	<p>CPF = £2.3 million</p> <p>VPF = £2.0 million PF = 5.0</p> <p>Proceed with works</p>
<p>Overall evaluation</p>	<p>Proposed improvement/refurbishment works will be carried out</p>	

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