The British Dam Society

Bruton Flood Storage Reservoir – Adopting a risk based approach to assessing spillway adequacy

A P COURTNADGE, Jacobs

SYNOPSIS Bruton Flood Storage Reservoir was originally built in the 1980s and was subsequently raised by 2m in 2009 and upgraded with a 50m long precast stepped-block spillway. The dam is now 14m high. A 10m high railway embankment crosses the downstream valley approximately 100m downstream of the dam, and in extreme floods, or if the underpass became blocked, tailwater could back up almost to the dam crest.

The 2009 design relies on this tailwater to protect the downstream face during extreme floods when the flank embankments overspill. The validity of this approach was reviewed in the recent Section 10 inspection with hydraulic modelling to assess various issues including the effect of the railway embankment breaching on tailwater levels, the time it takes for tailwater to establish, the increased scour risk at the location of the hydraulic jump and the effect of flow concentration due to mitres above the tailwater. The study found that the spillway did not comply with engineering standards and a risk-based approach was used to assess if the cost of upgrading the dam would be proportionate following ALARP principles.

This paper outlines the approach taken and the benefits of using a risk-based approach.

INTRODUCTION

The original Bruton reservoir was built in the early 1980s to alleviate flood risk in Bruton and is now operated by the Environment Agency. The dam was raised by approximately 2m in height and had a major spillway upgrade in 2009. One of the primary drivers for the scheme was that the peak velocity on the original spillway chute exceeded the recommended limit for the original surface protection, and an upgrade was mandatory as a measure in the interests of safety (MIOS) under the Reservoirs Act 1975. The current reservoir has a capacity of 842,500m^3 and is retained by a dam with crest 14m high above the original stream bed level.

The spillway adequacy was reviewed in advance of the routine Section 10 inspection report (S10) in 2023 by the author. This approach was requested by the Undertaker to minimise the likelihood of further studies being required as MIOS and to try and ensure that any MIOS measures resulting from the S10 were related to physical works, allowing them to better manage statutory deadlines. The review of spillway adequacy included a flood study, topographic survey, hydraulic modelling and a risk-based assessment to determine whether the cost of upgrading the dam would be proportionate. This paper summarises the approach and the findings.

Managing Risks for Dams and Reservoirs

DESCRIPTION OF THE RESERVOIR

Bruton dam is a homogenous clay embankment. The original dam was built from Forest Marble Clay and it was raised on the upstream side with Frome Clay. The drawings show a compacted clay cut-off up to about 4m deep, below the centreline of the dam crest, through the alluvial deposits, connecting to the underlying Frome Clay. Underdrainage is provided downstream of the cut-off trench, around the bed and banks of the original river channel, and as a collar around the culvert.

The dam is a Category A dam in accordance with 'Floods and Reservoir Safety' (FRS) (ICE, 2015). The spillway comprises a 48m long precast stepped-block spillway with its crest level at 75.2m AOD. It is a relatively rare form of spillway construction in the UK (Pether et al, 2009). In extreme floods water may also spill over the flank embankments. The downstream face of the flank embankment on the left side joins high ground, with a mitre contracting in towards the main spillway, whilst on the right-hand side the flank embankment runs up onto higher ground before turning upstream to follow a low embankment just upstream, and parallel to the railway. There is a training bund between the right edge of the spillway and the flank embankment creating a third mitre (Figure 1).

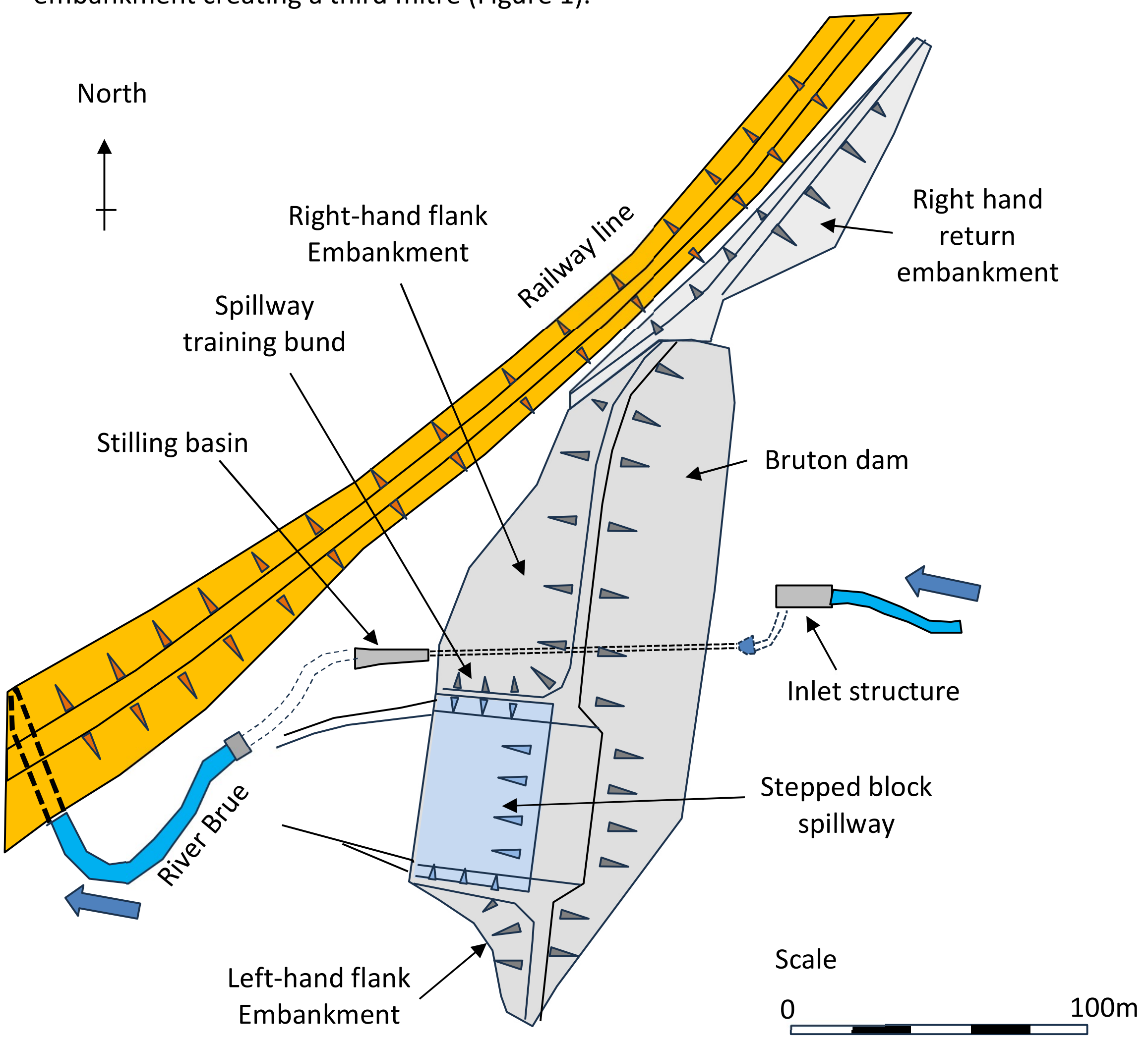

Figure 1. Schematic plan of Bruton Dam

The spillway was designed to store floodwater up to the 1 in 100-year flood event, which the designer predicted will be roughly equivalent to a 1 in 50-year event by 2059 allowing for climate change.

The outlet comprises a reinforced concrete inlet structure with debris screen connecting into a precast concrete culvert reducing from 1.8m to 1.67m diameter. The culvert discharges into a USBR Type III stilling basin. There is no bypass facility on the debris screen and it cannot be cleared from the dam crest.

A 10m high railway embankment crosses the downstream valley approximately 100m downstream of the dam, with low point of approximately 74.5m AOD, just below the spillway crest of 75.2m AOD. There is an underpass through the railway embankment for the watercourse and access track but in extreme floods, or if the underpass became blocked, tailwater could back up almost to the dam crest in extreme floods. The 2009 design relied on this tailwater and grass reinforcement was only provided on the upper part of the downstream face on the basis that the lower part of the face would be submerged. The rest of the downstream face and mitres comprise plain grass.

FLOOD ESTIMATES

A comprehensive flood study had previously been carried out in 2006 to inform the design of the spillway, which covered the Probable Maximum Flood (PMF) and T-year rainfalls with return periods up to about 200 years. The study did not consider the 10,000 year design event.

The study summarised previous estimates for the PMF dating from 1988 and 1996 which ranged from 300m^3/s to more than 500m^3/s. The 2006 study included a series of estimates broadly based on the methodology in the 1975 'Flood Studies Report' (FSR) (IoH, 1975) but with sensitivity analysis to consider historic flood events and changes to reflect concerns by Dr Colin Clark, a local resident and hydrologist who published several papers between 1996 and 2004 arguing that the FSR approach underestimates floods in southwest England (e.g. Clarke, 1996). The estimates varied from 143 to 514 m^3/s. The spillway design assumed a unit discharge of 5m^3/s/m over the spillway chute which equates to a PMF flood outflow of 270m^3/s, implying that this was the adopted design value and that the higher estimates were treated as sensitivity cases.

A further flood study was carried out by in 2023 primarily to estimate the 10,000-year flood but also to verify the previous PMF estimates and estimate the 1,000 and 100-year floods. In line with the earlier approaches the 2023 flood study also included a higher sensitivity estimate (PMF+) based on the probable maximum precipitation values from Clarke (1996). A hydraulic model was used to analyse routing of the flood event through the reservoir. Table 1 shows the various estimates of flood inflows and outflows.

Table 1. Flood estimates

Date	Author	Peak flood estimate (m^3/s) (inflow/*outflow*[1])				
		T-year	1,000	10,000	PMF	PMF+[2]
1982	Rendell Palmer & Tritton	50yr: 69; 100yr: 75			240	
1988		100yr:70/*20*			360	
1991	Rofe				365	
1996	Babtie	100yr: 32/*18*			322	
1996	Clarke				529	
2003	Babtie Brown & Root	100yr: 35/*18*			225	
2006	Black & Veatch	2yr to 200yr estimates: 17 to 68			143 to 514	
					270[3]	500
2023	Jacobs	100yr: 63	138/*119*	237/*216*	380/*364*	530/*521*

Notes.
1. Inflow in "roman" and outflows in "*italics*."
2. PMF+ is an upper bound estimate used for sensitivity analysis
3. Value adopted for wedge block design calculations

ASSESSMENT OF SPILLWAY CAPACITY

The assessment of spillway adequacy considered several aspects as summarised below.

Weir capacity and freeboard

A rating curve for the dam was generated using a hydraulic model. The culvert through the dam was assumed to be completely blocked which is reasonably foreseeable in large floods. The analysis indicated that the PMF stillwater flood rise would be 2.5m above the spillway crest and 0.55m over the crest of the flank embankment. The modelling indicated that the flank embankments would overtop by 90mm in a the 1 in 10,000-year Design Flood. It was therefore concluded that the spillway capacity did not comply with the standards recommended on page 7 of FRS unless the flank embankments were designated as auxiliary spillways.

Capacity of spillway chute (wedge blocks)

The original spillway design assumed a unit discharge of 5m^3/s which equates to a PMF flood outflow of 270m^3/s. There is no evidence that sensitivity analysis was previously carried out to consider the higher PMF estimates. The 2023 PMF estimate of 364m^3/s exceeds the original design flow by 35% and the PMF+ sensitivity estimate is nearly double the assumed design flow.

Guidance on the design of stepped block spillways is given in CIRIA Report 142 (CIRIA, 1997), which indicates that the mean block thickness would need to be up to 30% thicker to withstand the PMF+ flow. It is not known what factor of safety is incorporated into the design curve in CIRIA 142 but it could conceivably be less than 1.3 and it was therefore concluded that failure of the wedge block spillway could start to occur under the PMF flow, and could certainly occur in the PMF+ flow. To comply with full engineering standards (i.e. CIRIA 142), the wedge blocks would therefore either need to be replaced with thicker blocks, or the spillway widened to reduce the unit discharge.

Both the original design and the 2023 assessment were based on Figure 6.5 of the CIRIA 142 report which took into account the risk of a hydraulic jump occurring anywhere on the spillway face.

Velocities on main embankment face

The original design assumed tailwater, generated by the downstream railway embankment, would build up on the downstream side of the dam and grass reinforcement was therefore only included over the upper part of the slope, over an approximately 10m slope length. Investigations carried out as part of the inspection found that the reinforcement was only a two-dimensional plastic grid and was measured as being between 100mm and 135mm deep so is unlikely to be effective at reinforcing the turf (the author has found similar issues at other flood storage reservoirs).

The 2023 study included hydraulic modelling to test whether the original assumptions with regards to tailwater were valid, and in particular to:

- Confirm that tailwater would build up before the flank embankments overtopped.
- Assess how a breach of the railway embankment would affect the dam tailwater level.
- Consider concentration of flows and associated turbulence at the mitres on the left and right sides and along the right-hand spillway training bund.
- Consider the potential effect of a hydraulic jump on the downstream face.

Flood Modeller software was used to construct a 1D hydraulic model of the reservoir, the dam and spillway and the outlet culvert from the reservoir outlet through to the stilling basin. This 1D model was linked to a 2D TUFLOW HPC model to represent a 0.17km^2 area covering the downstream face of the embankment, the downstream valley, railway embankment, underpass and downstream weir. The 2D TUFLOW component has a grid size of 1m with topography informed by the 2023 survey and LiDAR DTM. It was assumed that the control structure and the River Brue culvert beneath the railway arch would block in the 1,000-year flood upwards. The following scenarios were modelled:

- Baseline: Railway embankment remains fully intact. This was modelled for the 100yr, 1,000yr, 10,000yr , PMF and PMF+ events to enable comparison with previous analysis.
- Railway embankment breach scenario. It was assumed that the railway would breach when the water level reaches three-quarters of the railway embankment height, based on Environment Agency guidance (EA, 2017). The breach was modelled as a vertical sided notch through the railway embankment. Two breach widths were considered; an initial breach width of 14m based on the Froehlich (2008) equation and an average breach width of 21m over the course of the flood event, on the basis that the breach

may double in width due to the continuing PMF flow for several hours after the initial breach. This was modelled for the 10,000yr, PMF and PMF+ events only, as the trigger water level for a breach was not reached in the 100 and 1,000year floods.

Peak velocities on the downstream face were inspected at key locations on the dam face and mitres. The analysis showed that in the peak velocities on the downstream face and mitres were within the limiting velocity for plain grass in the Design Flood but exceeded it in the PMF and PMF+ as illustrated by the colour coding in Table 2 below. The hydraulic modelling, and Table 2, does not explicitly represent turbulence, for example due to a hydraulic jump occurring on the face or the effect of flows plunging into the tailwater. The guidance in CIRIA 116 (CIRIA, 1987, page 36) recommends that where high tailwater would cause a hydraulic jump on the slope it may be advisable to provide heavier armour, or stronger restraint, than would otherwise be used to protect against high velocity flow alone. The risk of turbulence was considered separately and often meant that that the type of grass reinforcement required needed to be a level greater than indicated in the table.

Table 2. Peak velocities on grassed downstream face

Location	**Peak velocity (and durarion[1]) with 21m average railway breach (m/s)**				**Exposed height of face above tailwater[2] (m)**
	1,000yr	**10,000yr**	**PMF**	**PMF+**	
Left-hand mitre	No overflow	1.4	5.1 (2 hrs)	6.1 (2.5hrs)	3.8 to 4.2
Mitre with right-hand spillway cheek	No overflow	2.3 (1.2 hrs)	4.1 (2 hours)	5.5 (3 hrs)	3.8 to 4.2
Typical section of main embankment face	No overflow	0.8 (<1 hr)	3.7 (2 hours)	4.95 (2.8 hrs)	3.8 to 4.2
Right-hand mitre	No overflow	0.4	5.1 (2 hours)	5.7 (3 hrs)	3.8 to 4.2
Right hand return embankment	No overflow	No overflow	2.4 (<1 hr)	3.34 (2 hours)	4.3 to 4.6

Key (type of grass reinforcement required neglecting turbulence)[3 & 4]

Plain grass – poor cover	Plain grass – average cover	Plain grass – good cover	Open mat reinforcement e.g. Enkamat

Notes.

1. Duration is taken as the duration for which the flow exceeds 50% of the peak
2. Exposed height is the vertical height above the peak tailwater level. The range represents the different return periods. This indicates the approximate height over which grass reinforcement may be required. However, these values are based on the peak tailwater level and the exposed height is actually greater on the receding limb of the flood hydrograph with a maximum of 6.5m.

3. Colour coding indicates the type of grass cover/grass reinforcement required. Figure 9 of CIRIA 116 gives limiting velocities for different types of reinforced and unreinforced grass based on no damage occurring. In the PMF safety check flood some damage is acceptable so it would be acceptable to exceed the values in CIRIA 116 by some margin. There is limited guidance on what is an acceptable margin but a factor of 1.2 is commonly applied and was adopted for the PMF and PMF+ events. Section 2.3.3 of the Interim Guide to QRA (Brown and Gosden, 2004) suggests a factor of 2.0 on clay, and 1.0 on sand but this guidance is quite old and could be challenged by future Inspecting Engineers.
4. As discussed above, this table does not allow for turbulence which was considered separately and often meant that that the type of grass reinforcement required needed to be greater than indicated here.

Example output from the hydraulic modelling is shown in Figures 2 and 3. It was concluded that in order to comply with full engineering standards the downstream face and left and right-hand mitres would need to be reinforced with open mat reinforcement (e.g. Enkamat or similar) above around 70m AOD.

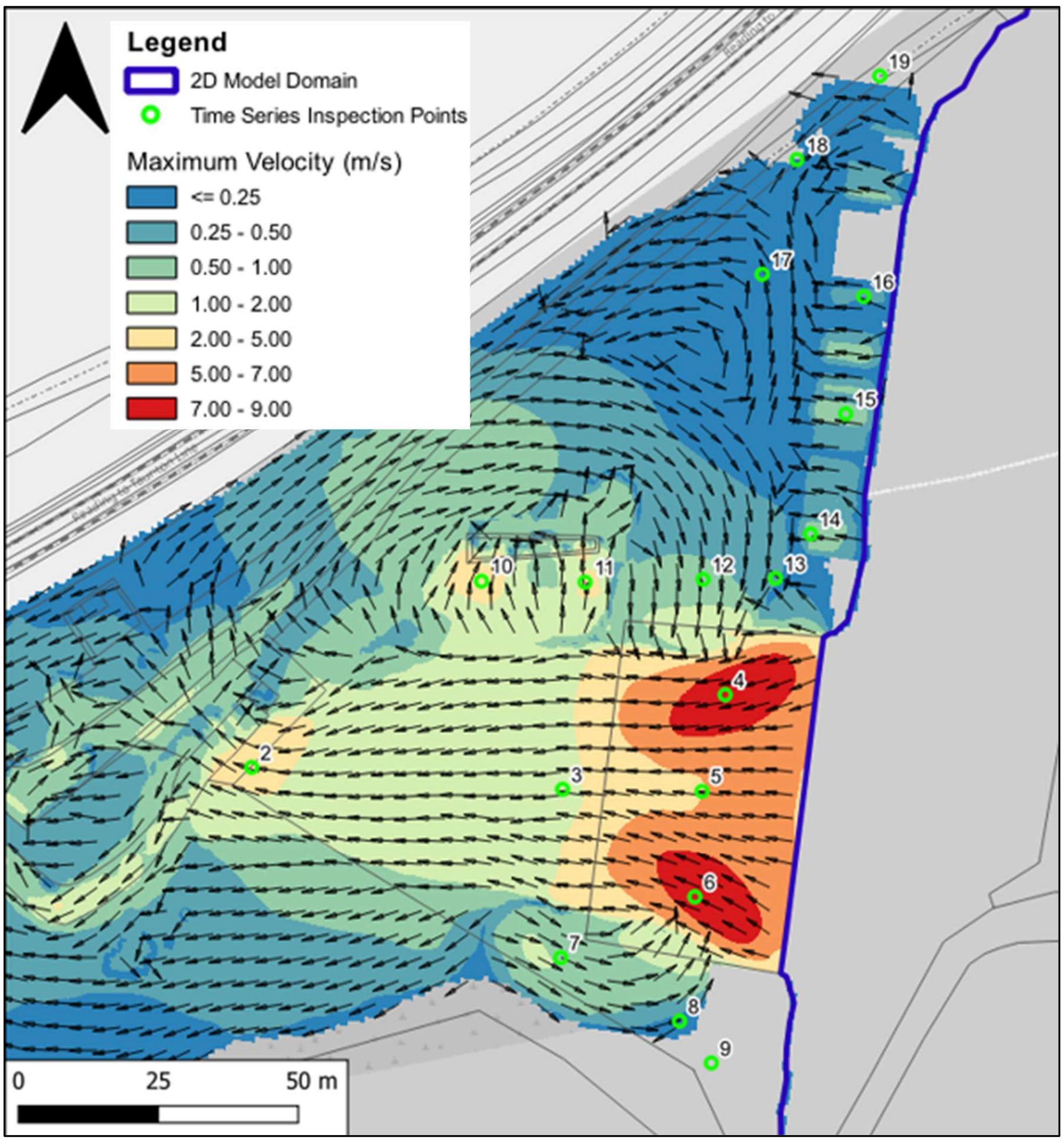

Figure 2. 10,000-year event maximum velocity grid with velocity point inspection locations

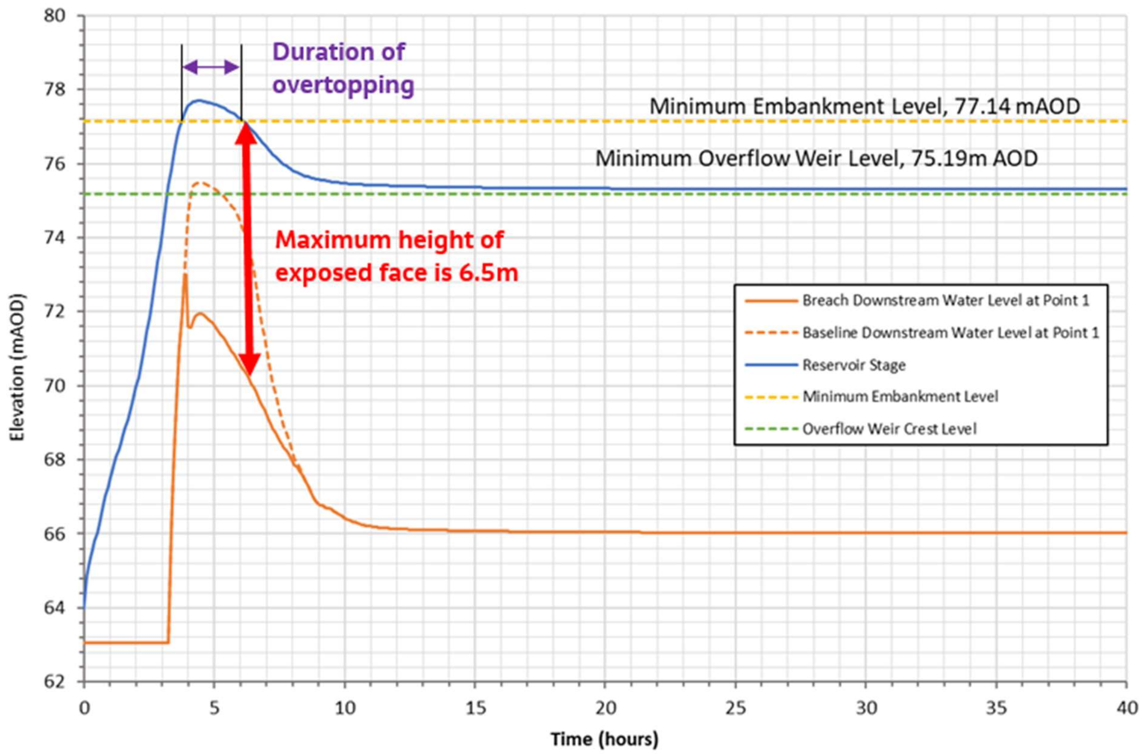

Figure 3. PMF Maximum Water Level with 21m wide railway breach

Summary of spillway adequacy

The assessment concluded that in order to comply with engineering standards, the whole of the downstream face of the main dam, including the three mitres, needs to be reinforced with open mat reinforcement and the spillway wedge blocks would either need to be replaced with thicker blocks, or the spillway widened to reduce the unit discharge.

ALARP STUDY

Where existing dams fail to meet the standards-based approach in FRS, the guidance advocates a risk-based approach to reduce risks to as low as reasonably practicable (ALARP). A study was therefore carried out to qualitatively assess the risks and help judge whether the cost of upgrading the dam would be proportionate to the level of risk reduction it would generate.

These economic calculations and sensitivity analysis were used as an aid to engineering judgement by an All Reservoirs Panel Engineer but were not in themselves the sole determinant. Such “risk-informed” judgment follows the principles set out in section 10.4 of the Guide to Risk Assessment for Reservoir Safety Management, Volume 2 (RARS) (EA, 2013).

Potential failure modes and current probability in failure

The study considered one overall failure mode (FM) for overtopping of the dam crest but considered the two ways in which failure could occur; scour of the grass face (FM1a), or failure of the stepped block spillway (FM1b). As discussed above, in both cases it was found that failure would be reasonably likely in the PMF event and very likely in the PMF+ event. Both failure modes would therefore need to be addressed to reduce the overall probability of failure.

FRS recommends that the PMF is assigned an annual exceedance probability (AEP) of 1 in 400,000. As discussed above, there is evidence that conventional flood estimation methodology underestimates floods in southwest England, hence in this case the average of the PMF and PMF+ estimates was assigned an AEP of 1 in 400,000. The PMF (364m^3/s) and PMF+ (521m^3/s) estimates were assigned AEPs of 1 in 100,000 and 1 in 900,000 respectively to give an approximately log linear relationship. On this basis, by interpolation, the annual probability of failure was assumed to be approximately 1 in 100,000.

Consequence of failure

The Environment Agency's Reservoir Flood Mapping (RFM) flood modelling summary sheet was obtained and used to assess the potential consequences of dam failure. The assessment was based on incremental consequences, i.e. the consequences over and above those predicted in an equivalent fluvial flood were the dam not to fail. The earlier 2005 dam break analysis was also reviewed but considered largely invalid because it predated the dam raising, assumed the railway would remain intact and excluded the higher PMF sensitivity estimates. Several limitations were noted with the RFM values:

a) Third party damages exclude damages to infrastructure. The RFM specification (EA, 2020) only covers damages to properties and does not cover the cost of repairing third party infrastructure. In this case additional allowances were added to cover possible compensation for damage to the downstream railway and a substation.

b) The consequences estimated by RFM were otherwise likely to be conservative because:

  - RFM would not have allowed for tailwater in estimating the peak breach discharge, so for all breach scenarios the peak breach flow is likely to be overestimated.
  - The RFM specification (EA, 2020,Section E.4.4) assumes a high erodibility dam but Bruton dam is built from an intermediate plasticity clay so is likely to be medium erodibility (see Table 10 of ICOLD, 2013).
  - The RFM modelling will not have considered the beneficial effect of the railway embankment and dam failing consecutively in terms of smoothing out the peak of the breach flows and allowing time for warning and evacuation. Indeed, the RFM does not allow for any warning or evacuation at all.
  - The fatality rates assumed by the RFM specification are based on a straight line best fit to observed deaths in flash floods and fluvial flooding in UK, with the data points shown on Figure 9.1 of the guide to risk assessment for reservoir safety (EA, 2013). It is noted that the USA use much lower fatality rates particularly where the Depth x Velocity is less than 1m^2/s (USBR, 2015), suggesting that the RFM fatality rates may be conservative.

Based on these considerations, upper bound, lower bound and best estimates were selected for the likely loss of life (LLOL) and cost of third-party damages. The best estimate for third party damages was 122% higher than the value from the RFM summary sheet based on point a) above and the LLOL was 24% lower due to point b).

Tolerability of current risk

The current societal risk of the failure was assessed by plotting the likely loss of life against the probability of failure on an FN-Chart. An example chart is shown in Figure 5 with the data points redacted due to their sensitive nature. The FN chart identifies three categories of risk, with definitions in Section 3.5.2 of RARS Volume 1 (EA 2013), as follows:

a) "*Broadly acceptable – risks people live with every day and which they regard as insignificant and not worth worrying about (for example, health risks associated with using mobile phones)*". No further analysis is normally considered necessary, although RARS (EA, 2013) does actually recommend that even then improvement works should be carried out unless the cost is grossly disproportionate

b) "*Within the range of tolerability*" (ALARP Zone) – "*risks that individuals and society are willing to live with the risks so as to secure certain benefits, provided that they are confident that they are being properly managed, and that they are being kept under review and reduced still further if and as practicable (for example, vehicular and airline travel).*" In other words, provided the risks are reduced to As Low As Reasonably Practicable (ALARP). The RARS guide therefore recommends upgrade works are carried out unless the cost of works is grossly disproportionate to the reduction in risk that would be achieved.

c) "*Unacceptable – risks that are generally thought by people as not worth taking regardless of the benefits.*"

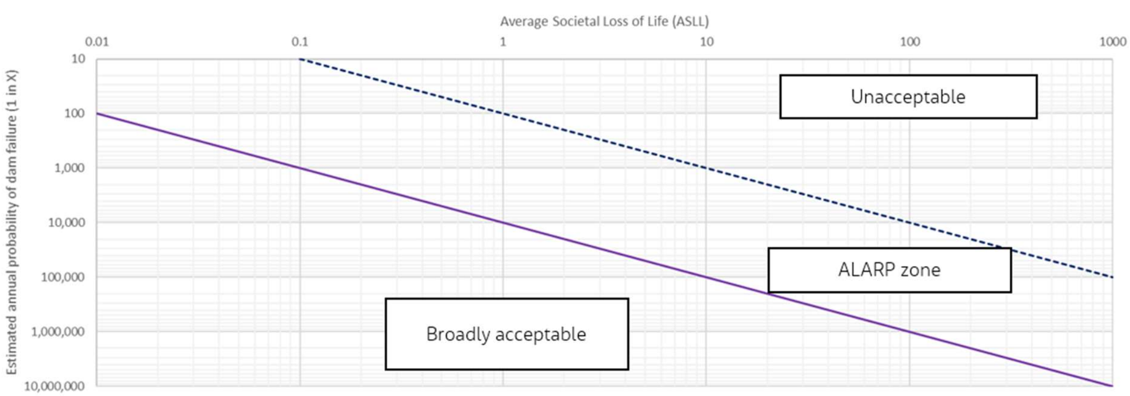

Figure 5. Consequence diagram for ALARP assessment (HSE, 2000 and Figure 9.2 of EA,2013)

In the case of Bruton, the best estimates for probability and consequence, plotted within the ALARP zone.

Assessment of whether upgrade works would be proportionate

In order to assess whether the cost of upgrade works is grossly disproportionate to the risk reduction they would generate, it is necessary to calculate the cost to prevent a fatality (CPF) for a particular upgrade option. The method is given in RARS (EA, 2013) and summarised as follows:

1. Multiply the reduction in annual probability of failure generated by a particular scheme by the Average Societal Loss of Life (ASLL), which is a statistical measure for the number of lives that could be lost in a catastrophic breach.
2. Discount this over a 100-year appraisal period to give a present value of likely savings in lives, using a factor of 57 (See Table 6 of Brown et al, 2014).
3. Multiply the reduction in annual probability of failure by the potential cost of third-party damages if the dam were to breach.
4. 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 57 as above.
5. The CPF is then calculated 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 CPF is then compared with the value of preventing a statistical fatality (VPF), with the current published VPF by the Department of Transport for road and rail schemes being around £2M.

To assess if costs are 'grossly' disproportionate, the HSE guidance (HSE, 2000) recommends applying a Proportion Factor (PF), i.e. the ratio of CPF/VPF, of between 2 and 10 depending on the overall probability of risk and accuracy of the estimates. A value of 10 was adopted for Bruton in recognition of the approximate nature of the risk and cost estimates. It therefore follows that ALARP was judged to be satisfied if the CPF is >£20million (i.e. 10 x VPF).

Candidate options

Four options to reduce the risk of failure mode FM1a were identified and two options to address FM1b as shown in Table 3. Any works would be required to address both failure modes to reduce the overall probability of failure and three combinations of these options were therefore assessed as shown in Table 4. Normally an ALARP study would assess a range of options varying in cost and level of compliance with full engineering standards to assess the level of expenditure that is proportionate, however in this case there are limited 'half-way' options, particularly in relation to FM1b.

Other potentially cheaper options which may partially mitigate the risks were discounted. For example, developing an evacuation plan or flood warning system was not considered appropriate given that the probability of failure is already 1 in 100,000 and because in the Hamstead Heath legal case it was ruled that evacuation plans should not be used as an alternative to carrying out required dam upgrade works (Hughes, 2016). It was felt that carrying out a more comprehensive dam break analysis, e.g. using LifeSim software, would be unlikely to materially affect the conclusions.

Table 3. Candidate options to reduce risk of flood overtopping failure

Option	Description	Assumed probability of failure after works[1]
Options to address FM1a – Erosion of grass face		
1	**Install open mat reinforcement** Reinforce the whole of the downstream face above approximately 70m AOD, including the three mitres, with an open mat grass reinforcement system such as Enkamat or similar. It would be good practice to include a kerb or crest beam to ensure uniform flow depth.	1 in 400,000
2.1	**Increase freeboard by 0.5m and build new emergency access route.** Raise main dam crest by approximately 0.5m. However, this would reduce the crest width to less than 3m which would limit vehicle access along the crest hence the need for a new access route. Extend the wedge blocks at the transition slopes on either side and the Dycel on the spillway cheeks.[2]	1 in 400,000
2.2	**Increase freeboard by 0.25m.** Similar to above but to mitigate the access issue described above, limit raising to 0.25m to ensure vehicle access remains possible along the crest.	1 in 300,000
3	**Create a formal auxiliary spillway.** Create an 80m wide auxiliary spillway to the right of the main spillway, by lowering the current crest by around 0.5m and reinforcing the slope with Grasscrete or Dycel. This would also reduce the unit discharge on the main spillway and therefore mitigate FM1b.	1 in 400,000
Options to address FM1b – Failure of wedge blocks		
A	**Replace wedge blocks with heavier blocks over whole spillway**	1 in 400,000
B	**Widen existing stepped block spillway by approximately 10m.** Whilst this would also reduce velocities on the grass face it would not reduce them sufficiently on its own, hence would still need to be carried out in conjunction with options 1-3 above	1 in 400,000

Notes,

1. Many of the options would actually pass the PMF+ flow which was assigned a probability of 1 in 900,000. However, for the purpose of the ALARP analysis a probability of 1 in 400,000 was adopted because that is the lowest probability normally considered in UK dam engineering.
2. This option would push 10% additional flow over the wedge block spillway which would exacerbate FM1b which in Table 4 would need to be covered by options A or B.

Table 4. Option combinations evaluated for ALARP

Option to address FM1a	Option to address FM1b	Assumed probability of failure after works	Approximate total cost
1	B (this is likely to be the most economic option)	1 in 400,000	£2million
2.2		1 in 300,000	£1.2million
3 (addresses both)		1 in 400,000	£3million

It was found that for all option combinations the CPF would be significantly over £20million. Using the best estimate consequences the CPF would be over £40million and even using upper bound consequences the CPF of the most favourable option combination (Option 2.2) was still £28million.

DISCUSISON

In its current state the likelihood of failure of the dam due to floods is of the order of 1 in 100,000 chance per year. Although this does not meet the engineering standard for a category A dam, in terms of a risk-based approach it is in the ALARP zone, where upgrades are only justified when the benefits of reduced likely loss of life outweigh the costs.

The cost to upgrade the dam spillway to meet engineering standards is of the order of £1 to £3 million depending on the option selected. ALARP calculations suggest that the cost to save a life is over £20 million per life saved, which based on guidance in RARS is grossly disproportionate to the risk reduction and not therefore warranted. The assumptions and uncertainties in the analysis have been reviewed and the above conclusion would remain valid even if worse case parameters were adopted.

A previous argument for accepting departure from full reservoir safety standards was that the total volume of water which could be released if the railway embankment and dam failed consecutively, would only be about 13% more than if the dam was not present, as shown in Figure 6.

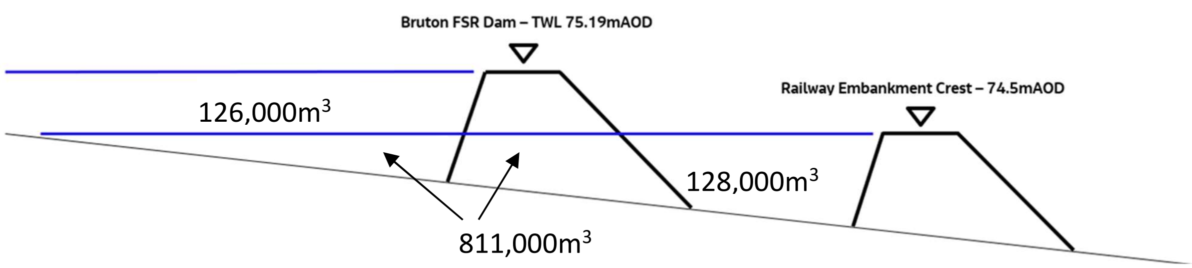

Figure 6. Illustration of escapable volume with and without the dam present

This argument effectively relates to the incremental consequences of failure but does not align with the guidance in FRS. Page 6 of FRS notes that "*in assessing the consequence of failure, it is the additional damage that would be caused if the dam failed under flood conditions compared with the damage caused by the flood were the dam not to fail*". There is a subtle difference in this wording compared to the previous argument which uses the base case 'if the dam was not there'. The incremental damages would be much more significant using the FRS base case as shown in Table 5

Table 5. Definition of incremental damages

	Previous argument	FRS wording
Wording:	'if the dam were not there'	'were the dam not to fail'
Base case:	Potentially large breach wave from 939,000m^3 storage volume behind railway	Relatively small breach wave from128,000m^3 volume in interspace between railway and dam
Incremental damages	Minimal	Significant

In summary it is not considered proportionate to carry out any spillway upgrade works.

CONCLUSIONS

The following conclusions are made:

- There is evidence that conventional flood estimation methodology underestimates floods in southwest England.
- The author has found on several occasions that grass reinforcement is not always as shown on the 'as built' drawings, often being too deep to be effective. A small trial pit is recommended to check the grass reinforcement during S10 inspections.
- When relying on tailwater to reduce the velocities on a spillway chute it is important to consider how quickly the tailwater will build up, whether a breach of a downstream embankment might affect the tailwater and the potential effect of a hydraulic jump on the downstream face.
- A risk-informed approach was used to qualitatively assess risks and help judge whether the cost of upgrading the dam would be proportionate to the level of risk reduction it would generate.
- This showed that the costs of upgrading the dam would not be proportionate to the risk, thus saving the Undertaker well over £1million.
- When using RFM output to assess the incremental consequences of failure it is important to recognise the limitations of this modelling
- Incremental consequences should be assessed against the base case "*if the dam did not fail*" rather than "*if the dam was not there*".

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