

The Reservoirs Act: Past, Present and Future – Papers

Click on paper title to jump to paper

Ref	Author(s)	Paper Title
Session 1: New Dams in the UK		
1.01	Blower & Golds	Potential applications of a ground risk management system to reservoir safety under the Reservoirs Act
1.02	Hughes	The use of site-won and non-standard fill materials in the creation of embankment dams
1.03	Lockett et al	Methodology for the use of Failure Mode Identification in the design of new reservoirs
1.04	Webb et al	Assessing the impact of the UK's glacial legacy on the design of dams – past, present and future
1.05	St John	Geochemistry considerations for the design of new earthfill embankment dams in the UK
1.06	Otten et al	Permeability testing across scales: Consequences for dam design in low permeability clay foundations
Session 2: International Examples		
2.01	Howard & Benn	Applying the Reservoirs Act 1975 to specks in the Atlantic Ocean
2.02	Miller	Strengthening Dam Emergency Preparedness: Lessons learnt from British Columbia, Canada
2.03	Yarritu-Sanchez et al	The Impact of the Tous dam Incident on Modern Dam Safety Regulation
2.04	Shaw et al	Design and Foundation Improvements at Yusufeli Dam to Accommodate Complex Geotechnical Conditions
2.05	Elías et al	Dam Safety Regulatory Compliance Challenges for Small Undertakers in Spain
Session 3: The new Reservoirs Act and Small Reservoirs		
3.01	Scott et al	Proportionality and reservoir reform
3.02	French et al	Sensitivity of reservoir breach flood mapping to erodibility assumptions
3.03	Morris et al	Changing Legislation; Practical implications, processes and policies for the retrospective registration of 10,000m ³ – 25,000m ³ reservoirs
3.04	Shaw & Holland	Large Raised Reservoir 10,000m ³ Threshold: Challenges Faced by New Undertakers
3.05	Terrel & Bugler	Help! I own a reservoir
Session 4a: Flood Detention Reservoirs		
4.01	Grogan et al	Adaption and Improvements to Leigh Flood Storage Reservoir
4.02	Wood et al	Investigation and management of the 2024 Simpson overflow incident
4.03	Yuste Zaballos et al	Westmoor Reservoir: Lessons learned through design and construction
4.04	Benn et al	Cove Brook Flood Storage Reservoir – upgrading to meet modern reservoir safety standards
Session 4b: Concrete Dams & Monitoring		
4.11	von Loggeren & Smith	Underwater Enabling Works for a Scour Culvert Gate Overhaul at Leixlip Dam
4.12	Webster et al	Rehabilitation of the Underdrainage System at Cow Green Reservoir

Ref	Author(s)	Paper Title
4.13	Coombs et al	Capturing and assessing the conditions in concrete dam drainage galleries using novel camera technology and AI image processing techniques
4.14	Henschel et al	Permanent seepage monitoring using thermal measurements at Toddbrook Reservoir following extensive improvement works
4.15	Bell & Knox	Revitalising Resilience: Modernising 17 Service Reservoirs for a Sustainable Future
4.16	Hoppe et al	From Data Management to Integrated Dam Safety: DAMDATA, a Digital Platform Approach
Session 4c: Risk and Emergency Exercises		
4.21	Hoppe et al	Implementing a National Risk-Informed Framework for Dam Safety: Lessons Learned from Spain
4.22	Brown et al	Benefits of Portfolio Risk assessment – a case study on flood detention reservoirs
4.23	Cubbage & Harding	Testing Reservoir Emergency Arrangements: Lessons from a Post-Incident Exercise at Towcester
4.24	Stuart et al	Climate Change Impacts on Flood Risks – What Can Dam Owners Do?
4.25	Down et al	Inspections of Small Raised Flood Storage Reservoirs
4.26	White et al	Application of the Piping Toolbox 2024 to Historic Embankment Dams: Canal & River Trust Case Studies
Session 5: Hydraulics and Hydrology		
5.01	Bell & Watson	Hydraulic Harmony: Balancing Safety and Supply at Lough Island Reavy
5.02	Golds et al	Protecting Wales' largest reservoir: dam safety and lasting resilience at Llyn Celyn
5.03	Holloway et al	Townhead Reservoir – The Real-World Impact of Changes to UK Flood Hydrology
5.04	Barnard et al	Bewl Drawdown Improvements Scheme: Design, Delivery and Commissioning of a Large-Scale Siphon System
5.05	Piedra et al	Weir discharge coefficients for reservoir flood studies: values, methods, and effect on reservoir flood levels
5.06	Erskine et al	Optimising reservoir flood safety and storage capacity: Comparative design of labyrinth and ogee weirs
5.07	Stanway	Welford Reservoir Venturi Primed Flexi-Hose Syphon - A UK First?
Session 6: Geotechnics		
6.01	Butler et al	Core? What core? Lessons from Pre-1840 Embankment Dams on Locating Puddle Clay Cores
6.02	Holden et al	Witley Court: Incident Response and Interim Repair
6.03	Archibald & Holland	Emergency Response at an Historic Reservoir in North Wales: Emergency Drawdown and Discontinuance
6.04	Taylor et al	Learning from success and failure – embankment dams founded on high-plasticity over-consolidated clays
6.05	Power et al	Overview of the Project Pollaphuca Erosion Remediation Works detailing reservoir erosion remediation works and associated lessons learned
6.06	Koracevic & Potts	The factor of safety in finite element analysis of embankment dams
6.07	Mapperley	Some considerations in assessing hydraulic uplift risk for large reservoirs on overconsolidated clays



The British Dam Society

Potential applications of a ground risk management system to reservoir safety under the Reservoirs Act

T BLOWER, *formerly* Mott MacDonald (retired)

S GOLDS, Mott MacDonald

SYNOPSIS

Major infrastructure projects frequently harbour significant ground-related risks, and reservoirs constructed with earth-fill embankments all the more so. For such projects, the ground risks make up a major component of the overall project risk profile. Accordingly, a ground risk management system may often be appropriate.

Anglian Water is planning the construction of two new raw water storage reservoirs in the East of England to address future deficits in public water supply. These are massive infrastructure projects, classified as Nationally Significant Infrastructure Projects (NSIP), and each includes a large raised reservoir, built on and with earth materials.

As part of the Engineering Leadership initiative within these projects, engineers and geologists have developed a ground risk management system that provides a framework for how ground-related risks are to be identified, managed and mitigated throughout the lifetime of the project.

In this paper, the authors reflect on how a formalised system, supported by appropriate risk registers and risk management plans, and instigated from the outset of a large reservoir project, may contribute to the assurance of dam safety by supporting Reservoirs Act 1975 regulatory requirements.

INTRODUCTION

The link between ground conditions and risk has been long established in major civil engineering projects. One seminal work in the UK ground investigation industry was entitled “Without site investigation ground is a hazard” (SISG, 1990). This document explained how a large proportion of cost and time over-runs on heavy civils projects were attributable to uncertainty in ground conditions. However, few guidance documents provide a detailed suite of tools and procedures for managing such risks throughout the lifespan of a project.

Probably the most well-developed system for managing ground risk around major infrastructure is that used in the highways sector, as shown in document CD622 Managing Geotechnical Risk (National Highways, 2020).

Potential applications of a ground risk management system to reservoir safety under the Reservoirs Act

This document sets out a series of processes and documents, for phases including investigation, design, trials, construction and maintenance, all with the objective of identifying, mitigating and managing geotechnical risks. No such system has yet been established for large raised reservoirs.

GROUND RISKS IN CONTEXT

Stages of Project Development

To provide a framework for discussion on how risks change with time, it is helpful to refer to the project stages referred to in the Institution of Civil Engineer's (ICE) Manual of Geotechnical Engineering (MOGE), (Burland et al., 2012), as summarised, with some simplification, in the following table:

Table 1. Simplified Stages of a Major Project (after Burland et al., 2012).

No.	Stage	Typical Activities
1	Preparation	Investigations, environmental studies, preliminary design, planning activities.
2	Design	Design development (in various stages), design of environmental mitigations and enabling works.
3	Pre-construction	Contract development, procurement activities, implementation of enabling works and some environmental mitigations.
4	Construction	Construction of the works; continued environmental mitigations
5	Operation, Use	Operation, use, maintenance and monitoring.

The stage of "De-commissioning" could be added, but for reservoirs, whose lifespan is measured in centuries rather than decades, predicting suitable risk mitigation actions so far in advance is not considered helpful.

Which risks?

Technical experts, including ground engineering professionals, are normally concerned with the principal risks of technical adequacy, health and safety, cost, time and environmental impact. However, at project development stage many other matters are also at risk or at least highly uncertain. Project promoters and managers are concerned about project commercial viability. For a project requiring approval through the Development Consent Order route, planning risk is a major feature. Procurement risk is also a major consideration, particularly where a Special Purpose Vehicle is established to plan, design, build and operate, and even fund, the asset. Nevertheless, 'Ground Risk' remains a significant component of the overall project risk, requiring both mitigation and management throughout the project cycle.

Risk Mitigation Tools

Given the variety of risks to be managed, it is unsurprising that a variety of risk mitigation tools are used by ground engineering practitioners engaged on major projects. The following paragraphs discuss some of these.

Site Investigation (SI) and Ground Investigation (GI)

Uncertainty in the ground conditions at a project site is a key ground risk. All construction projects are characterised by a flurry of site investigation activity in the early stages. This includes desk-based studies and intrusive ‘ground investigations’ that explore the make-up of the ground directly.

Whilst there is some guidance on the scale of intrusive investigations that are required, for example in the Eurocodes, for larger projects these rules of thumb do not apply so well. Legitimate questions remain:

- How much GI is appropriate, and when?
- Who will benefit and who should procure the next stage of GI?
- When are other risk mitigation actions more suitable than more GI?

For larger projects, then, a more nuanced approach is required and this is often linked to the perceived risks. Such approaches focus on targeting GI activity on the items of perceived greatest risk, to enhance GI cost-effectiveness. Arnold et al. (2020) describe such an approach used on HS2.

Ground Modelling

Significant advances have been made in ground modelling in recent decades, helping practitioners to make the most out of limited factual data. Suitable modelling allows specialists to be explicit about where uncertainty exists and what the bounds of that uncertainty are. Parry et al. (2014) introduced three levels of ground models, namely Conceptual, Observational and Analytical Models. Of these, Observational Models have advanced greatly, allowing manipulation of large data sets and providing 3-D visualisation tools.

Design Interventions

The design approach also has a key influence on the level of risk carried forward through the planning and design stages of a project. For example, if there is concern about the reservoir embankment foundation strength, one approach would be to undertake more and more detailed investigations, to allow refinement of the assessment of the field-mobilised shear strength. Another approach would be to assume the worst credible shear strength in all analyses. Whilst ‘safe’, this latter approach would make the design very cautious, with implications for cost, programme, and so on. This also introduces the concept of an Assumptions Register, which is discussed later.

Potential applications of a ground risk management system to reservoir safety under the Reservoirs Act

Practical trials

Notwithstanding any amount of GI and testing, there can remain significant uncertainties about how the ground will actually behave in practice. This is because GI methods can only test small samples/areas of the ground and thus its ‘mass behaviour’ remains a matter of conjecture. It is thus common practice to undertake practical earth-moving trials to fill this knowledge gap.

Compaction Trials are the least costly, involving excavation of the earth materials, which are compacted in layers using a variety of equipment. By varying certain parameters, such as moisture condition, layer thickness, and number of passes, it is possible to assess an optimal compaction procedure, which can support and inform construction planning.

However, only a **Trial Embankment** will provide information on the response of the embankment foundation to the embankment loading. Typically these are much more expensive, involving the construction of a large scale section of embankment. This type of trial gives an indication of how the material(s) behave(s) under excavation, handling and placement, and also of the likely foundation lateral and vertical displacements, and global stability. Of great importance is the rate of pore water pressure dissipation in the foundation, as this affects stability and the allowable rate of construction, and is particularly difficult to estimate from laboratory tests on (small) samples.

Contractual interventions

All construction contracts contain mechanisms for sharing risks. For example, in the New Engineering Contract (NEC), a Compensation Event arises if the Contractor “encounters physical conditions” for which it “would have been unreasonable to have allowed”. Thus, theoretically, the more GI data included in the Site Information, the less likely it is that a compensation event based on unreasonable ground conditions can be notified.

The projected Contract cost is affected by the apportionment of risks between the parties. The more risk the Contractor is expected to carry, especially in relation to the scale of the risk when looking at a new reservoir, the higher will be the likely risk allowance and most probably, the projected contract costs. A particularly onerous contract may thus make a project unaffordable. On the other hand, Clients quite reasonably seek a degree of cost certainty before committing to such a large investment.

In certain sectors, Geotechnical Baseline Reports (GBRs) are used to provide a transparent mechanism for sharing ground risk (Davis et al., 2023). These have achieved widespread usage in tunnelling (ITA, 2025), and uptake is now extending into other areas, such as reservoir construction. Practice reveals, however, that whilst useful they are not necessarily a panacea.

The timing of interventions

Another key consideration is the timing of mitigation interventions. For a large new reservoir construction project, numerous phases of GI can be envisaged. Not only does the question “how much GI is enough?” arise, but so too does the more difficult question “how much GI is enough for now?”. Some GI costs could be abortive if project details change during planning.

Similarly compaction trials and trial embankments would ideally be carried out as early as possible to provide design benefits as early in the design process as possible. However, they may have to be repeated if the intended beneficiary (a contractor or designer) does not feel they can place legal reliance on such data. This would push towards a trial embankment carried out later by the contractor that is ultimately selected for the works.

THE ANGLIAN WATER RESERVOIRS

Anglian Water, which serves nearly seven million customers covering 14 counties across the East of England and Hartlepool, is planning for the construction of two new raw water storage reservoirs, referred to as Fens Reservoir and Lincolnshire Reservoir, to address projected future deficits in public water supply. The reservoirs will be Nationally Significant Infrastructure Projects (NSIPs) seeking consent through Development Consent Order (DCO) applications.

Both reservoir projects involve the construction, commissioning and operation of a new non-impounding reservoir with an approximate capacity of 55 million cubic metres (Anglian Water, 2025). The embankments will be created from locally excavated soil materials. The projects also include the infrastructure required for water conveyance and treatment.

During the early design development for the Fens and Lincolnshire reservoirs, substantial investigations and early design work helped identify a number of ground risks, large and small. These were captured on a risk register from the earliest stages, but it was recognised that a structured approach to their evaluation, mitigation and management was required. Project leaders therefore requested that a Ground Risk Management Strategy be devised, firstly to capture what had been achieved to date in terms of ground risk mitigation, but also to map out what further steps would most likely be appropriate at future stages of the projects.

ANATOMY OF A GROUND RISK MANAGEMENT SYSTEM (GRMS)

The discussion below is taken principally from the work undertaken for the above Anglian Water reservoirs. In this paper the acronym GRMS stands for the Ground Risk Management System, of which the Strategy is just one part.

Potential applications of a ground risk management system to reservoir safety under the Reservoirs Act

Ground Risk Management Strategy

The principal component of a Ground Risk Management System is the Strategy itself, which should set out the high-level aims of the System, on a number of horizons, such as pre-DCO, pre-construction and in service. It should also present the framework within which the various risk mitigation interventions are to be implemented. The Strategy should also recognise the contractual context as set out in the Commercial Strategy, for example.

Ground Risk Registers

Geotechnical practitioners are well versed in spotting potential risks in the ground, based on the geological and geotechnical conditions revealed, or in recognition of a lack of GI information. Early geotechnical deliverables often include risk assessments, or hazard identification lists, based on desk studies and early ground investigation activities. These are a good start, but often the degree of risk is difficult to judge in the early stages as project details may not yet be fixed, and the design approach not yet fully developed. More developed risk registers will typically follow after some progress has been made in design development, and these typically include assessments of impacts, as well as likely mitigation actions, their time-lines and owners.

Whilst myriads of potential ground risks may be recorded on a risk register, their significance, individually, may be limited and thus not of great import to strategic decision-makers. On the Anglian Water projects it has been useful therefore to introduce a hierarchy of risk registers, with four levels, from the lowest, L4, concerning individual technical matters, to the highest, L1, concerning strategic to the enterprise. Project leaders can thus focus on the latter, whilst technical specialists consider the detailed risks below.

Assumptions register

A natural corollary of the **Risk Register** is an **Assumptions Register**, since every risk identified is based on a set of assumptions. More conservative design assumptions will reduce the residual risk, whilst more adventurous ones will increase it. Similar examples can be found in the contractual planning sphere, for example in the sourcing of key materials, as well as in planning matters, for example in what landscape mitigations are likely to be required. If these assumptions are wrong, costs and programme time may increase. An assumptions register or log is thus a key document.

Ground Risk Management Plans

To pull all of these elements together, in the interests of advancing the goals of the Strategy, it may be useful to have a Risk Management Plan (RMP) for each ground risk subject area. Such plans can set out the various issues

related to a specific subject area, refer to existing data, and current assumptions, and identify the associated potential risks. The RMPs for each subject area then set out the likely mitigation actions over the lifetime of the project, from planning, through to operation and maintenance.

For the Anglian Water reservoirs, this was seen as a useful exercise, allowing all discipline leads to then comment on the assumptions, the risks identified and the proposed mitigations, and providing a framework for discussion on the basis of which it would be expected that the proposed mitigations, or 'action plan' might evolve with time as the project progresses.

GRMS for the Anglian Water reservoirs

In order to capture in one place all of the elements that could be considered 'ground risk', the ground risk management plans encompassed a very broad range of matters in the ground, from peat and hydrogeology to UXO and utilities as well as purely geotechnical matters, as shown in Figure 1.



Figure 1. RMP's supporting the Ground Risk Management Strategy

This broad scoping of ground risks has the advantage that when gathered up within the highest level Risk Register, the documentation clearly sets out what is and is not included in the strategic-level "ground risk".

The work undertaken for Anglian Water in 2025/26 culminated in a first draft of the whole GRMS for the planned reservoirs. One of the key features of this system is that it needs to be integrated. A change to one component of the system necessarily leads to a review of, and adjustments to, other parts of the system, see Figure 2.

Potential applications of a ground risk management system to reservoir safety under the Reservoirs Act

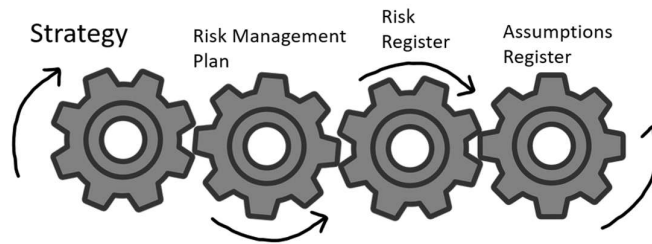


Figure 2. Representation of an Integrated Ground Risk Management System

THE RESERVOIRS ACT 1975

In a review of the effectiveness of the Reservoirs Act 1975, Warren (2026) states that the UK “has an excellent record in dam safety and there has been no loss of life from dam failure since 1925”. However, the serious reservoir safety incident at Toddbrook reservoir in 2019 (Warren and Bennett, 2021) led to a review of the legislation, adding to previous calls for legislative reform and a more proportionate and flexible regime for reservoir safety.

How does the Reservoirs Act 1975 (RA75) deal with ground risk?

RA75 does not deal explicitly with ground risk – rather it manages all risks by requiring that qualified dam engineers assess, monitor and mitigate all reservoir safety hazards including those in the ground. Section 7(6) does refer specifically to geological and geotechnical information being included in the as-constructed information in the Annex to the Certificate of Efficient Execution of the Works (CEEW) which is part of the ‘dam memory’. Although not explicitly enshrined in legislation, as a part of their duties under the legislation, qualified engineers regularly carry out quantitative and/or qualitative risk assessments related to ground conditions, for example for internal erosion risk, using The Piping Toolbox 2024 (UNSW, 2024) or for other reservoir safety risks in line with RARS guidance (Defra, 2013).

Application of a GRMS in the context of the RA75

In light of the above, the authors have considered what the potential benefits of a formal written GRMS (including all of its components as discussed above) might be for the implementation of the Reservoirs Act as it is, but also in relation to possible forthcoming legislative reforms in England and Wales (Warren, 2026).

The following table summarises the aspects where potential benefits of a GRMS have been identified through the project lifecycle. They apply primarily to new-build reservoirs, but parallels can be anticipated with major remedial works to existing reservoirs or for ongoing assessment of risk during the operational phase of older reservoirs.

Table 1. Potential Application of a GRMS to the functioning of the Reservoirs Act 1975

Position Holder	Reservoirs Act Section	Requirements of the Act	How a GRMS can be applied
Construction Engineer (CE)	6(1)	The CE is “employed to design and supervise the construction or alteration”. This applies to new reservoirs (or for a change in top water level or reuse of abandoned reservoirs).	Management of changing technical risks (together with their budgetary and contractual implications) interfacing with reservoir safety decisions. This is relevant as the programme develops from concept design through detailed design to construction, commissioning and operation. The GRMS recognises that the perceived ground risks continue to evolve throughout the project lifecycle – they are not fixed upon completion the first ground investigation!
CE	6(2)-6(4)	CE provides a certificate to confirm that the reservoir can be partially or wholly filled.	Along with surveillance, monitoring and other important information, the GRMS can provide support and evidence to assist the CE with certification.
CE	7(6)	This section of the Act specifically requires inclusion of geological and geotechnical information in the as-constructed information to be included in the Annex to the CEEW.	Residual risks should be captured through the construction handover information (CEEW, H&S file, as-built drawings etc.). For major projects it will be beneficial for the GRMS, with relevant residual risks highlighted, to be included in the as-constructed information in the same way that, in the context of health and safety legislation, a Designer Risk Assessment is passed on. These documents together form the beginnings of the ‘dam memory’.

Potential applications of a ground risk management system to reservoir safety under the Reservoirs Act

Position Holder	Reservoirs Act Section	Requirements of the Act	How a GRMS can be applied
CE	6(5) 7(1)-7(4)	Until the final certificate is provided, the reservoir is under the supervision of the CE, after which the SE takes over. This is usually about 3 years after the end of construction.	The GRMS and associated RMP's effectively track the mitigation of ground risks through the design and construction phases of the project and record residual risks that will need management in the operational phase. The GRMS is developed alongside other plans such as the Instrumentation and Monitoring Plan and the Inspection and Maintenance Plan, and it thus supports a structured transfer of residual risks into operational management systems. In the future, under proposed legislative reform, there may be a requirement for a Reservoir Safety Management Plan (RSMP) and Reservoir Safety Case (RSC) (Warren, 2026) and the GRMS would then support these documents in the assessment of the relevant failure modes. Residual ground risks, such as leakage, settlement and earthworks defects will be managed through the processes outlined in operation and maintenance plans (potentially to be included in the proposed RSMP in the future), all informed by the life-long tracking of potential ground risks as recorded in the GRMS and associated RMP's.
CE	7(5)	The CE shall consider matters to be watched by the SE.	
Supervising Engineer (SE)	12	Supervision of large raised reservoirs including emergency planning	
Inspecting Engineer (IE)	10	Inspections. Periodic safety review. Potentially in the future: the Reservoir Safety Case.	
Qualified Civil Engineer (QCE)	10(6) + SI No. 1896 §6(1)(b)	Supervision and certification of reservoir safety works.	
Undertaker / Reservoir Manager	All sections	All requirements	The GRMS would be held by Undertakers or Reservoir Managers as part of the suite of documents which assist with the management of the asset through its life cycle.

DISCUSSION

From this initial review, there do seem to be benefits to having a formal GRMS to support the implementation of the Reservoirs Act 1975. These benefits appear only to be enhanced if the proposed new legislative regime is implemented. The following paragraphs capture some discussion points.

- The GRMS primarily supports the development phases of the reservoir project and provides key inputs into the Reservoir Safety Case (definition of hazards, failure modes and mitigations) and the Reservoir Safety Management Plan (monitoring, surveillance and operational controls).
- Risk management plans do not replace professional judgement by engineers with the appropriate skills, knowledge and experience – rather they are tools available to those managing reservoir safety risk.
- Using a risk-based approach there should be a sliding scale of complexity of GRMS with complexity of project. This approach fits together with the proposed RSMP and RSC within a risk-based approach to reservoir safety management. So for smaller projects, a GRMS could be captured in a single, simple document. On others, e.g. minor remedial works or low hazard dams, a formal GRMS may not be appropriate if the inputs and cost are disproportionate to the risk.
- A more detailed review of how a GRMS might support the RSMP and RSC should be undertaken when the newly proposed reservoir safety reform is finalised.

SUMMARY & CONCLUSIONS

A formalised Ground Risk Management System was developed in the early stages of design for two proposed new reservoirs in the East of England. The associated documents have provided a platform for discussion of the mitigation of ground risk throughout the lifetime of each of the projects.

Whilst useful for these specific projects, consideration has been given to whether this type of formalised GRMS could have benefits for wider implementation of the Reservoirs Act 1975. The initial conclusion is that it could. The potential benefits of a formal GRMS should be considered again once the Government have settled on the details of their proposed reforms to the Reservoirs Act 1975.

ACKNOWLEDGMENTS

The authors would like to thank Anglian Water for permission to publish this paper, and to colleagues in Mott MacDonald for assistance and support.

Potential applications of a ground risk management system to reservoir safety under the Reservoirs Act

REFERENCES

- Anglian Water Ltd (2025) *A proposed reservoir in the Fens; A design refinement report*; Anglian Water, Peterborough, October 2025.
- Arnold C et al. (2020) *Managing uncertainty in ground conditions in design on a megaprojects where ground investigation is highly variable*; <https://learninglegacy.hs2.org.uk>.
- Burland et al., (2012) *Manual of Geotechnical Engineering, Chapter 7 – Geotechnical risks and their context for the whole project*, pages 59-73
- British Standards Institution (2024). *BS EN 1997 : 2024 Eurocode 7 - Geotechnical Design (all parts)*.
- Davis J et al. (2023). *Geotechnical baseline reports: a guide to good practice. CIRIA Report C807*, CIRIA, London, January 2023
- Defra (2013), *Guide to risk assessment for reservoir safety management*
- Hill T et al. (2014) *A Guide to the Reservoirs Act 1975, Second Edition*. Institution of Civil Engineers, London.
- ITA (International Tunnelling Association) (2025). *NEC4: Guidance on NEC and Underground Works*. NEC Contract.com, ITA-AITES, June 2025
- Mapperley N et al. (2026) Some considerations in assessing hydraulic uplift risk for large reservoirs on overconsolidated clays. In *The Reservoirs Act: Past, Present and Future – Proceedings of the 32rd Biennial Conference of the British Dam Society*. Keele, 2026.
- National Highways (2020). *CD622 Managing geotechnical risk*; part of Design Manual for Roads and Bridges.
- Parry S et al (2014) Engineering geological models: an introduction. IAEG Commission 25. *Bulletin of Engineering Geology and the Environment*, 73, pp689-706.
- University of New South Wales (2024); *The Piping Toolbox 2024; Methods for Estimating the Probability of Failure of Embankment Dams by internal Erosion and Piping*.
- Warren A L (2026). Development of Improved Reservoir Safety Management Systems for England & Wales; In *Symposium “Water, Energy, and Society: The Evolving Role of Dams in a Changing World” – 94th ICOLD Annual Meeting – Guadalajara, Mexico, May 2026*.
- Warren A L and Bennett N (2021). *The Toddbrook Reservoir spillway incident and emergency response*. ICOLD Symposium, New Delhi.



The British Dam Society

The use of site-won and non-standard fill materials in the creation of embankment dams

M W HUGHES, AtkinsRéalis

SYNOPSIS

The planning and construction of new large, raised reservoirs in southern and eastern England has renewed attention on the challenges of designing and constructing dams using high plasticity, over-consolidated clays. Across parts of the United Kingdom, the bigger challenge is simply finding suitable clay at all. Where good clay is not readily available or economically viable to import, engineers can look to use site-won or non-standard fill materials.

This paper will present three cases where site-won or non-standard materials were used to build flood attenuation embankments (designed as large, raised structures). It will look at the key challenges in their design and the selection and use of the materials.

INTRODUCTION

Low-permeability cohesive soils, particularly if clay-rich and of intermediate plasticity, are widely regarded as suitable for dam construction.

New reservoirs planned or under construction in southern and south-east England are making use of locally borrowed high plasticity over-consolidated bedrock clays as embankment fills. Their use presents challenges related to moisture conditioning and compaction; pronounced shrink–swell behaviour which increases susceptibility to desiccation cracking; and elevated sulphide contents which affect sourcing of filter sands and pose risks to concrete. As a result, the use of bedrock clays requires enhanced quality control, zoning restrictions, and careful assessment of long-term durability. Despite these challenges, their use in UK dam construction is well-established.

What if a suitable cohesive soil is not locally available and importing large quantities of fill is impractical or prohibitively costly? In such cases, engineers would need to consider the use of non-standard fill materials that fall outside those traditionally regarded as suitable for dam construction.

This paper presents three such examples in which flood storage embankments, designed by AtkinsRéalis as large, raised structures, were constructed using non-standard fill materials that were either site-won (chalk, colliery spoil) or imported (granular fill, Glacial Till). Key challenges, including design considerations, material selection and construction control

The use of site-won and non-standard fill materials in the creation of embankment dams

are discussed. Although it contains some technical material, the paper is not intended to function as a technical guide; interested readers are encouraged to consult the referenced national and international guidance.

AVAILABLE GUIDANCE ON THE SELECTION AND USE OF FILL MATERIALS

Guidance – United Kingdom

UK embankment dam guidance can appear prescriptive in its preference for clay-rich fill materials with defined properties but CIRIA (1996) mentions the economic and environmental advantages of using locally sourced fill even if it is not clay-rich. The guide recognises that marginal or otherwise unsuitable fills can often be incorporated through appropriate engineering design. The guide accepts that, although the design solutions may increase total fill volumes, construction complexity and supervision requirements, they can remain cost-effective compared with importing higher-quality materials.

Similarly, Environment Agency (2016) recognises that design may need to accommodate difficult materials where local constraints apply. Some challenging materials are mentioned in the report, including the following which are the subject of this paper:

- *Chalk* as certain grades can have very poor properties, be susceptible to deterioration when handled and be frost-susceptible.
- *Highly permeable materials* which require consideration of leakage.
- *Glacial Tills* as they can be inherently variable with particle sizes ranging from coarse sands and gravels to silts and clay.

Environment Agency (2016) suggests that the coefficient of permeability of fills used to form flood storage embankments should ideally be $\leq 10^{-7}$ m/s, but higher permeability values might be tolerable through careful design.

In summary, whilst clay-rich fill materials are preferred, UK guidance accepts that material selection is driven by a balanced assessment of availability, performance, constructability and whole-life cost.

Guidance – United States

The US Army Corps of Engineers (2004) states that most soils may be used provided they are insoluble and substantially inorganic; however, clays with liquid limits exceeding 80 % should generally be avoided. This criterion would include London Clay in certain locations, particularly within the eastern London Basin and parts of Kent and Gault Clay.

US Bureau of Reclamation (1987) indicates that most soil types are acceptable for use in homogeneous dams and as cores in zoned dams. These

include low and high-plasticity silts and clays, as well as silty or clayey sands and gravels, provided the fines content exceeds 12%. Clean sands and gravels are considered unsuitable due to their high permeability. The guidance also advises against using organic soils for major portions of the dam or core, and explicitly identifies peaty soils as unsuitable.

In summary, US embankment dam guidance is more explicitly performance-focussed, encouraging the cost-effective use of a wide range of locally available fill materials, subject to appropriate investigation, engineering design and construction.

CHARACTERISTICS OF THE EXAMPLE SCHEMES

The example schemes are located throughout the UK and vary in both the materials used in their construction and the engineering measures adopted to support the use of those materials. However, they share several common characteristics:

- they are earthfill embankment dams constructed for flood storage;
- they are large, raised structures as defined by the Reservoirs Act 1975;
- they are built on-line and are normally empty;
- they were designed to overflow, either at a lowered section of the dam crest or across the full crest width; and
- all earthworks were specified and controlled using the Specification for Highway Works (SHW), currently National Highways (2025).

This paper does not present detailed case studies of each scheme. Instead, simplified cross-section details are provided and discussion is limited to those components directly relevant to the subject of this paper. These components are identified by an asterisk (*) on the simplified cross-sections.

EXAMPLE SCHEME 1 – SOUTH WEST ENGLAND

The dam was constructed using locally available and site-won **CHALK**. It is straight in alignment, about 165m long with a maximum height of 3.9m.

Material Sources

The economic benefits of the scheme depended on the use of chalk fill from an existing stockpile located nearby and under the same land ownership.

The ground investigation identified a thin layer of Head Deposits overlying Chalk at the dam site. The Chalk was weathered and unstructured to depths of up to 4m below ground level. The dam was designed to be founded on the weathered chalk, with a cut-off trench extending into the underlying

The use of site-won and non-standard fill materials in the creation of embankment dams

structured (intact) chalk. Excavated arisings were reused as fill within the cut-off trench.

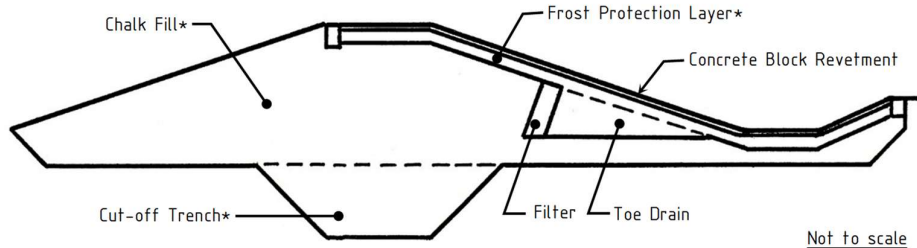


Figure 1. Example Scheme 1 - Simplified cross section of chalk fill dam

Dissolution

Chalk is a soluble carbonate rock containing constituents that dissolve to form solution features along vertical and sub-vertical discontinuities. Once reworked and compacted, chalk fill lacks a continuous fracture network and therefore has no inherent preferential flow paths. However, the chalk fill–conduit contact surface may act as a potential pathway for water movement and dissolution. The likelihood of dissolution pipes developing was considered to be very low, as the process typically occurs over geological timescales rather than within the design life of the dam, and the site lies in a predominantly dry valley where runoff is neutral to slightly alkaline.

Frost susceptibility

Frost heave tests confirmed that the locally stockpiled and site-won chawks were frost susceptible. The risk of expansion displacing the concrete block revetment was considered and mitigated by adding an imported granular underlayer to protect against saturation and insulate the chalk. A thickness of 450mm was adopted in accordance with guidance for highway construction (CIRIA (2002)). To ensure compliance with the hydraulic limits set out in CIRIA (1987), an upper limit of 1×10^{-5} m/s was specified for the coefficient of permeability (k) of the compacted granular fill.

Material Classification

CIRIA (2002) describes the classification of chalk using four grades to characterise in-situ chalk based on its structure and weathering state, ranging from Grade A (intact, well-structured chalk) to Grade D (unstructured chalk). Grade D chalk is further subdivided, using the suffixes 'c' or 'm', where 'c' denotes clast-dominated material exhibiting granular soil behaviour, and 'm' denotes material dominated by a chalk matrix exhibiting cohesive soil behaviour. Following investigations, locally stockpiled chalk

was classified as Grade Dm, while the site-won chalk was classified as Grade Dc (during the main works, it was subsequently reclassified as Grade Dm).

Material Specification and Use

The use of the above classification system cannot be relied upon as the same chalk can change grade as it breaks down during excavation and compaction. SHW classifies chalk as 'Class 3' and Highways Agency (1995) advises that chalk should be specified as a whole, allowing the adoption of method compaction. However, experience from the Channel Tunnel Rail Link (CTRL), documented in CIRIA (2002), led to the informal sub-division of Class 3 chalk into behavioural classes from Class 3E (fill instability unlikely to occur during compaction) to Class 3H (moisture sensitive and not suitable for railway embankments). Source testing found the available chalks to fall within sub-classes 3E and 3F (with properties given in Table 1.).

Table 1. Chalk material properties

Property	Class 3E	Class 3F1	Class 3F2
Intact dry density (Mg/m ³)	>1.70	1.45 – 1.70	1.45 – 1.70
Moisture content (%)	<25	25 – 28	20 – 25
Saturated moisture content (%)	-	<25	<28
Chalk Crushing Value	<3.8	<3.8	<3.8

End product compaction requirements were also set in accordance with the CTRL criteria, corresponding to 10% air voids (or less) at field moisture content, which were developed to limit the potential for inundation collapse settlement of the chalk fill. A series of compaction trials were undertaken to confirm the appropriate compaction method, leading to the use of a padfoot roller to condition the fill for final compaction by a smooth roller. Early trials also investigated lime stabilisation to reduce moisture content, as construction was originally programmed to commence in autumn and continue through winter. However, it was instead agreed to delay the start of the works to the spring of the following year.

During construction, achieving the specified compaction was difficult and frequent reworking was required. Spring rainfall caused wet chalk to heave, rendering it unworkable. More problematic, was the unusually hot and dry summer that led to the rapid drying of any stockpiled chalk. Although the Contractor sought to retain moisture by lightly compacting stockpile surfaces, this was only partially effective and increased compactive effort was often required. Where chalk had been stored for extended periods, and moisture content fell below 20%, water was added by rotavator mixing.

The use of site-won and non-standard fill materials in the creation of embankment dams

Water addition, placement and compaction were undertaken as a single operation and proved effective in consistently achieving the target density and air voids with reduced compactive effort and minimal reworking.



Figure 2. Inspection of cutoff trench, note the prepared stockpiles of excavated chalk intended for re-use

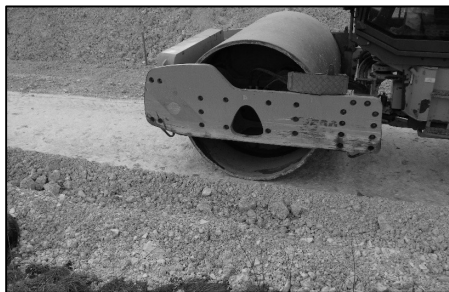


Figure 3. Trial compaction of locally stockpiled chalk



Figure 4. Compaction using both padfoot and smooth rollers

EXAMPLE SCHEME 2 – NORTH EAST SCOTLAND

The dam was constructed using imported **GENERAL GRANULAR FILL**. It is straight in alignment, about 250m long with the typical height of around 2m, locally increasing to a maximum of 5m at the impounded watercourse.

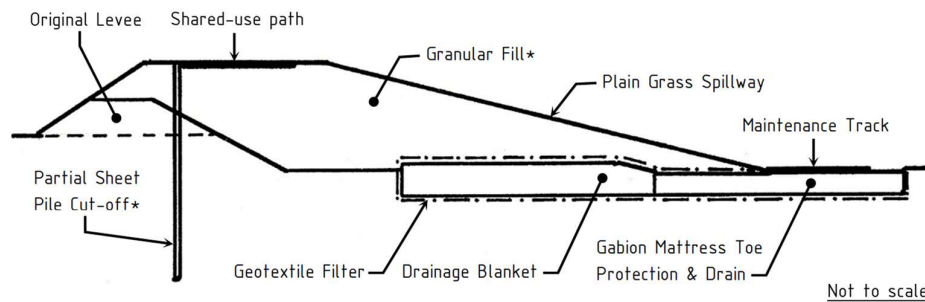


Figure 5. Example Scheme 2 - Simplified cross section of granular fill dam

Material Source

The embankment was originally designed to be constructed using cohesive Glacial Till, which was known to be widespread across the region. However, the main contractor confirmed that such material was not readily available at the time of construction, with no commercially available sources nor any concurrent construction projects able to provide suitable or reliable quantities of surplus material. Consequently, it was agreed that the embankment design would be revised to incorporate granular fill, sourced from a commercial quarry located approximately 25km from the site.

Control of seepage

The drainage blanket shown in the simplified cross-section was included in the original design (assumed cohesive till), to manage potential seepage beneath the embankment. It was intended to prevent seepage emerging at the downstream toe, which could adversely affect the performance and stability of the grass spillway. This drainage blanket was retained in the revised design, with a partial interlocking steel sheet pile cut-off added to limit seepage through the embankment. The sheet piles incorporated a factory-applied sealant within the clutches to improve water-tightness. Seepage modelling was undertaken to confirm the adequacy of the system in controlling pore water pressures and seepage flows.



Figure 6. Installation of sheet piles



Figure 7. Completed partial cut off

The use of site-won and non-standard fill materials in the creation of embankment dams

Material Specification and Use

The granular fill was well graded and, with 12% to 15% fines (non-plastic), complied with the requirements for SHW Class 1A General Fill. The maximum particle size was limited to 75mm (rather than the standard 300mm) to ensure that the piles could be driven readily without obstruction.

To ensure compliance with the hydraulic limits set out in CIRIA (1987), an upper limit of 1×10^{-5} m/s was specified for the coefficient of permeability (k) at a target density of 95% Standard Proctor maximum dry density (MDD). Constant head laboratory tests measured a maximum permeability of 3×10^{-6} m/s at 72% MDD, indicating that the specified upper limit would not be exceeded at the target density. In-situ sand replacement tests confirmed an average of 100% MDD was achieved, with no values below 95%.

EXAMPLE SCHEME 3 – SOUTH WEST WALES

The dam was constructed using locally available **UNBURNT COLLIERY SPOIL** (shoulders) and locally sourced **GLACIAL TILL** (core). It is straight in alignment, about 145m long with a height at the spillway crest of 9.3m.

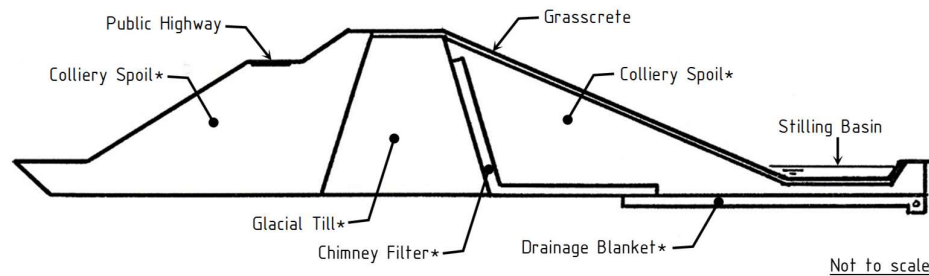


Figure 8. Example Scheme 3 - Simplified cross section of zoned dam

Material Source

The proposed dam was located within the boundary of a disused colliery site. The general fill for the embankment was originally intended to be wholly sourced from a colliery spoil tip located against the valley hillside about 300m downstream of the dam site. The use of site-won fill was not required to economically justify the scheme as this was already proven based on fully importing fill. The driver was instead both practical and environmental:

- A nearby primary school severely restricted the timing of deliveries to site so fill could only practically be delivered during school holidays.
- Using site-won material would be inherently more sustainable and with a reduction in carbon cost due to less traffic movements.

CIRIA (1998) suggests colliery spoil can be used for a variety of hydraulic engineering works including river embankments and impoundment dams. SHW permits the use of burnt and unburnt colliery spoil in highway works. BSI (1985) states that spontaneous combustion of unburnt spoil can be discounted if compaction is undertaken in accordance with SHW.

It is important to differentiate between burnt and unburnt spoil as they present different benefits and challenges in terms of use and performance:

- Unburnt spoil is a good general earthworks material. Some particle breakdown during compaction helps it achieve a well-graded, mechanically stable fill of low permeability but this behaviour prevents it from being used as a selected fill.
- Burnt spoil is generally stronger and more granular, making it suitable for higher-grade applications such as sub-bases but it is more likely to present issues such as high sulphate content and frost susceptibility.

On consideration of the properties, it was decided that the available colliery spoil would be suitable for use if it was unburnt.



Figure 9. Colliery spoil being sourced as fill for the embankment dam

A copy of the Coal Authority tip report was obtained. It confirmed that the tip had previously been relocated from the hilltop to the valley side after the closure of the colliery. It mentioned high temperatures within the original tip, suggesting it might contain burnt spoil. The ground investigation revealed the material to be black, relatively homogenous and generally granular, with 10% to 20% fines (non-plastic) and relatively consistent moisture content. The description suggested the spoil was unburnt as it was black, rather than the reddish colour typical of burnt spoil. Mass loss on ignition test results, together with particle density and chemical test results, supported the conclusion that the spoil was unburnt.

The use of site-won and non-standard fill materials in the creation of embankment dams

Control of seepage

The unburnt colliery spoil was of low permeability however, it was recognised that its non-plastic nature made it susceptible to cracking arising from differential settlement. This was mitigated in design by introducing:

- A chimney filter to intercept seepage through any developed cracks and prevent internal erosion.
- A 'core' of modified spoil, with added bentonite clay to increase plasticity and thereby reduce the potential for cracking.

A large-scale earthworks trial was undertaken during the enabling works phase. The trial investigated variations in bentonite content, mixing techniques and methods of compaction. The suitability of the resulting material was assessed based on repeatability, achieved dry density and permeability. The trial outcomes were successful, enabling the development of an end product specification tailored to the works.

The enabling works contractor was not awarded the main works contract. The successful contractor subsequently proposed an alternative materials strategy, comprising the use of the colliery spoil for the embankment shoulders and the importation of a cohesive Glacial Till for the core. The till was surplus material from another project located approximately 4km from the site, for which they were also the contractor.

The proposed Glacial Till comprised a clay of intermediate plasticity with 20% to 30% fines, including a 10% to 15% clay fraction. The designer reviewed the suitability of this till and concluded that it was acceptable. This was justified on the basis that the design already incorporated measures appropriate for mitigating the risk of seepage resulting from possible material variability (EA (2016)), namely a relatively wide core and inclusion of a chimney drain. In addition, the designer recommended the introduction of a hold point to allow placed layers to be inspected for areas of aggregation, which were to be excavated and replaced if identified.

The Glacial Till was imported to site in bulk during a single school holiday period and was subsequently protected with weighted plastic sheeting until the contractor was ready to place the material.

Material Specification and Use

The specification required all earthworks materials to be compacted to a minimum of 95% of the Standard Proctor maximum dry density (MDD). In addition, fill placed in the embankment shoulders was required to achieve air voids of less than 10%, while fill placed in the core was to be compacted wet of optimum, thereby further limiting air voids to less than 7%.



Figure 10. Chimney drain between core (left) and downstream shoulder

As noted in published guidance, the colliery spoil performed well as an earthworks material, although it was susceptible to degradation if worked under wet conditions.

There were initial difficulties working with the Glacial Till, as both compaction trials and the first layers consistently failed to meet the specified compaction criteria. The designer was therefore asked to review the testing approach and the results obtained. This review concluded that while the testing was undertaken in an appropriate manner, the outcomes did not consider the influence of the oversized fraction, defined as material retained on the 20mm sieve and above. As a result, a practical testing protocol was developed to explicitly allow for the effect of the oversized fraction.

The protocol involved determining the particle density and water absorption of the oversized fraction and comparing these with the particle density measured for the whole sample using the glass jar method. This enabled the moisture content of the finer fraction (passing the 20mm sieve) to be used to derive a corrected moisture content–dry density relationship, with adjusted values of maximum dry density and optimum moisture content. For assessment of material acceptability and compaction compliance, the percentage dry mass of the total sample retained on the 20mm sieve was determined and plotted against the corrected moisture content–dry density curve, allowing both moisture content and field dry density results to be assessed in a more representative and meaningful manner.

Once the protocol was established, use of the Glacial Till became more straightforward, with non-compliance rates reducing to levels more typical of routine earthworks. There were only a few instances which required fill

The use of site-won and non-standard fill materials in the creation of embankment dams

to be removed and replaced due to aggregation and these were limited to small, isolated areas.

CONCLUDING REMARK

Overall, these examples schemes demonstrate that non-standard, site- and locally-won materials can be used successfully where their behaviour is properly understood and accommodated. By recognising material limitations and aligning the design with the intent of published guidance, designers and contractors can make effective and responsible use of available materials while maintaining performance and safety objectives.

REFERENCES

- BSI (1985). *BS 6543: Guide to use of industrial by-products and waste materials in building and civil engineering*. British Standards Institution, London.
- CIRIA (1987). *Report 116: Design of reinforced grass waterways*. Construction Industry Research and Information Association, London.
- CIRIA (1996). *Report 161: Small embankment reservoirs*. Construction Industry Research and Information Association, London.
- CIRIA (1998). *C513: The reclaimed and recycled construction materials handbook*. Construction Industry Research and Information Association, London.
- CIRIA (2002). *C574: Engineering in chalk*. Construction Industry Research and Information Association, London.
- Environment Agency (2016). *Design, operation and adaptation of reservoirs for flood storage*. SC120001/R. Environment Agency, Bristol.
- Highways Agency (1995). *Design Manual for Roads and Bridges, Volume 4 Section 1 Part 1 (HA 44/91) Geotechnics and Drainage. Earthworks: Design and preparation of contract documents*. Department for Transport, London.
- National Highways (2025). *Specification for Highway Works, Clause CC 601: Earthworks. Manual of Contract Documents for Highway Works*. Department for Transport, London.
- US Bureau of Reclamation (1987). *Design of small dams. 3rd edition*. US Department of the Interior, Denver.
- US Army Corps of Engineers (1995). *Engineering manual EM 1110-2-1911: Construction control for earth and rock-fill dams*. USACE, Washington, DC.



The British Dam Society

Methodology for the use of Failure Mode Identification in the design of new reservoirs

S LOCKETT, Mott MacDonald

A WILDEE, Mott MacDonald

J FOSTER, Mott MacDonald

SYNOPSIS

Population growth and climate change in the UK are placing increasing pressure on water resources, necessitating the development of Strategic Resource Options (SROs) to ensure long-term supply security. Among these options are proposals for new reservoirs. At the earliest phases of design, the process of Failure Mode Identification (FMI) and risk management strategy of As Low As Reasonably Practical (ALARP) can be used as a tool considering all stages from the construction, commissioning, operation, maintenance and decommissioning of the assets. The method can be used to justify and set priorities for Reservoir Safety Design Principles that can be bound into the planning stages and tender reference design of new reservoirs. This paper describes how the methodology of FMI as described in the Environment Agency's "*Guide to Risk Assessment for Reservoir Safety Management*" (RARS, EA 2013) could be applied to the design of new large-raised reservoirs.

BACKGROUND ON RARS

Summary of background principles

Historically, reservoir safety was assessed against prevailing engineering standards. This approach was recognised as flawed as it can be reactive, overlook failure modes, and misdirect investment priorities (Brown, 2025). In May 2013, the Environment Agency published the "*Guide to Risk Assessment for Reservoir Safety Management (RARS)*" (EA, 2013). This framework, predominantly developed for the assessment of existing dams, supports a risk-based assessment accounting for the diversity of dam types, forms, and physical settings.

The RARS process is structured as follows:

- a) Pre-assessment to define context and objectives
- b) Risk identification through failure mode identification
- c) Risk analysis considering probability and consequences
- d) Risk evaluation against tolerability criteria

Methodology for the use of Failure Mode Identification in the design of new reservoirs

Failure Mode Identification (FMI) is central to reservoir safety management. It proactively identifies and analyses potential failure points, addressing “what could go wrong” and, in the case of existing dams, considering whether the failure modes are credible and significant and, if so, scoring the likelihood of progression to failure. In the case of new dams, the same technique can be applied to consider mitigation strategies focused towards designing out potential failure modes such that they are very unlikely to be credible and significant.

Benefits of risk assessment for large reservoir programmes

To address water supply challenges and support housing development the UK government is supporting the development of Strategic Resource Options (SROs). Several key projects are being proposed including the construction of the first major new reservoirs in the UK in 30 years including:

- Havant Thicket Reservoir: Located in Hampshire, this reservoir will supply to the towns of Portsmouth and Havant.
- White Horse Reservoir: Situated in Oxfordshire, this reservoir aims to supply water across the South East.
- Fens Reservoir: situated in Cambridgeshire being developed in partnership by Anglian Water and Cambridge Water, this reservoir is part of a larger water supply project across the region.
- Lincolnshire Reservoir: Developed by Anglian Water and situated in Lincolnshire.
- Cheddar 2 Reservoir: A new reservoir near Cheddar and one of three flagship projects being undertaken by West Country Water and Environment (WCWE) to secure water resources across the region for generations to come.

These projects are part of the government's Plan for Change, which aims to build 150 major infrastructure projects this Parliament to create new jobs and drive economic growth. The new reservoirs are expected to provide significant benefits, including reducing the impact of droughts and managing river levels in wetter periods.

An SRO is a regulatory and funding designation with projects being delivered to address regional or national supply-demand deficits. Projects need to be enrolled in the Regulator's Alliance for Progressing Infrastructure Development (RAPID) programme for development funding and regulatory oversight. This is a systematic programme where the project needs to advance via a set of 'Gates' which are all reviewed by the regulator.

Under the Planning Act 2008 (as amended by the 2019 Order), a new reservoir would also be classified as a Nationally Significant Infrastructure Project (NSIP) if either the storage is more than 30 million m³, or the deployable output is more than 80 Megalitres per day (ML/d). For NSIPs, the consent is via a type of planning permission called a Development Consent Order (DCO). This method of consenting is unique and currently untested for new reservoirs and applying the FMI methodology can assist in providing clear justification to support this process.

Risk assessment is now a core element of reservoir engineering, moving beyond purely deterministic compliance. Carrying out FMI in accordance with RARS at the planning stage provides clear value for large reservoirs by embedding a structured understanding of risk at a stage when design influence is the greatest.

At an early design stage, the FMI process can be applied and aligned with best practice to ensure the design team systematically identifies how a reservoir could fail and to inform associated mitigation measures. Large-raised reservoirs impound water above the surrounding natural ground level so, by their very nature, pose a potential high consequence in the event of a breach failure. It is therefore essential that reservoir design applies very high standards such that the overall risk posed by the structures is extremely low. Failure mode identification can be done in advance of quantifying breach risk and Likely Loss of Life (LLOL), by considering the scale of the reservoir (impounded height and volume) and location (proximity to densely populated areas) to indicate the likely consequence of any failure.

The value of FMI therefore lies less in numerical outputs and more in ensuring all credible failure mechanisms—internal erosion, slope instability, overtopping, and foundation failure—are identified early and explicitly addressed through design, with mitigation aligned to ALARP (As Low As Reasonably Practical) principles.

Undertaking FMI at the pre-planning stage allows mitigation to be incorporated while the scheme remains flexible, as key decisions on layout, embankment form, spillway provision, and outlet configuration are still being defined. Early identification of failure modes enables inherently safer solutions such as improved zoning, enhanced drainage and filter arrangements, and increased hydraulic capacity, rather than relying on later refinement or retrofit.

FMI also improves the robustness and defensibility of the scheme during planning and regulatory scrutiny. It provides a clear, traceable demonstration that dam safety has been systematically addressed, which is

Methodology for the use of Failure Mode Identification in the design of new reservoirs

particularly important for large reservoir schemes such as SRO or NSIP projects given their scale, visibility, and regulatory complexity. This builds confidence among regulators, panel engineers, and stakeholders, and demonstrates alignment with ALARP expectations, reducing the risk of challenge or post-consent redesign.

Finally, FMI supports better whole-life outcomes, addressing failure mechanisms early and reducing the likelihood of costly design changes, construction challenges, and operational constraints. As such, FMI enhances safety, programme certainty, and long-term value by ensuring proportionate and justified risk reduction.

INTRODUCTION OF FAILURE MODE IDENTIFICATION

Reasons for incorporating Tier 1 RARS

Tier 1 RARS (RARS, EA 2013) with the associated FMI process is widely used around the reservoir safety industry and is based on similar high consequence projects. It provides a justifiable approach to incorporating and protecting key safety design measures and potentially provides a basis to challenge for those safety measures which could be perceived as not providing value or benefit to the community as they may not be well understood by people who are not dam specialists.

The FMI process can not only be used for the planning application, but could be used in the design and construction development as a tool to: -

- provide a robust methodology to develop design principles
- support or challenge design change – especially by non-reservoir safety commercial teams
- manage the risk register of the project.

Methodology

The risk assessment for a new reservoir design can initially be focused on the critical FMI phase. The RARS guide was developed for the assessment of the risk posed by existing dams, whether they would be considered as credible and significant and to help inform decision over the proportionality of remedial work to reduce the risks to ALARP.

Table 7.2 of the guide could be used when developing the design of a new reservoir. Each potential threat to the reservoir can be considered along with how these could progress to failure. For failure to occur, an initiating mechanism must arise from a specific threat, followed by progression of a

failure mode that could result in breach, uncontrolled release of water, and downstream damage or loss of life.

THREATS: events or conditions that may result in a hazard and initiate a potential failure. They can be Internal or External.

EXTERNAL THREATS include: Floods (or uncontrolled inflow for non-impounding reservoirs), Winds/waves, Adverse weather, earthquake, or actions of people (including rapid drawdown).

INTERNAL THREATS include: deterioration due to factors such as stress changes from consolidation and/or cyclic loading and may occur within the dam body, foundation, or at interfaces with structures.

PROGRESSION (Failure Mode): the mechanism by which a hazard could cause dam failure (i.e. an uncontrolled release of retained water or a condition where the dam can no longer safely retain water due to loss of structural integrity) and may result from one or more issues leading to loss of freeboard or stability and eventual breach. Examples include scour or saturation of the downstream face reducing stability, settlement from consolidation or internal erosion causing overtopping.

Rather than assessing credibility and significance (as a in traditional risk assessment), the focus should then be on identifying design measures that would tend to make each potential failure mode either not credible or not significant. This could then be categorised into three aspects:

- Physical solutions
- Design assumption
- Operational and Maintenance requirements

The initial FMI document should then be tested and refined through workshops with design teams, technical specialists, Construction Engineer and key stakeholders.

Outputs could inform key reservoir safety documents such as:

- Reservoir Safety Execution Plan
- Key design assumptions affecting reservoir safety
- Mandatory reservoir safety requirements

These documents can then inform or be appended to Contract documents and reference designs at a later stage.

Reservoirs are unique structures with a potential asset life stretching far beyond usually specified design lives. The average age of UK reservoirs is

Methodology for the use of Failure Mode Identification in the design of new reservoirs

greater than 100 years with some canal dams stretching back to the late 1700s. The FMI assessment should therefore consider the asset life requirements of the reservoir and (considerably) shorter design lives of individual components such that design decisions are not only considering what failure modes may develop soon after construction, but what could potentially develop over the entire asset life of the structure.

In addition to the long-term operational safety, the assessment should also consider hazards associated with other critical stages including construction and first filling.

Key findings of FMI – Examples from New Reservoirs

Many of the new reservoirs under consideration are ‘non-impounding’ reservoirs. These are reservoirs that do not impound a watercourse and are instead predominantly filled by pumping. They often require embankments to be formed around the entire perimeter where the topography is not suited to tying into existing areas of high ground.

When considering the threat posed by floods or, in the case of a non-impounding reservoirs, uncontrolled inflow (which may be the more critical case), the FMI process can be used to consider whether there is justification for an overflow, where the normal operational water level should be set, and how it is designed to safely pass peak inflows.

Assessment of the type of spillway based on flow velocity alone may permit an unreinforced surface but identification of the potential vulnerability to damage from other factors (e.g. debris lines) may provide clear justification for the inclusion of something more robust such as modular concrete reinforcement.

Adverse weather, particularly prolonged dry conditions exacerbated by climate change, poses a potential risk of desiccation of large earth embankments. Identification of this failure mode could lead to inclusion of additional protection such as a capillary layer above the core and/or the downstream face.

Risks from human actions should also be considered. A key emerging threat to SROs including reservoirs is cyber-attack, particularly unauthorised operation of drawdown valves. Although controlled, the discharge capacity may be significant and, depending on the surrounding topography, could result in localised flooding. The design therefore could assume that remote operation will either be excluded or safeguarded against cyber intrusion.

Human activity also affects long-term operation. In the case of Nationally Significant Infrastructure Projects (NSIP), the reservoir site may be required

to deliver 'wider benefits' to the community. These could include a combination of recreation, education, tourism and Biodiversity Net Gain (BNG) features.

Delivering this wider benefit could involve recreational buildings located on the embankments. One potential failure mode could relate to uncontrolled service penetrations through the dam core, so designated crossing points in the form of utility corridors could be considered, aligned perpendicular to the crest and situated above the expected flood level.

Landscaping may be proposed on the downstream slopes to allow the reservoirs to integrate better with the surrounding area. There could be several potential failure modes that these features could introduce without adequate mitigation such as roots encroaching the core or being uprooted in storms with potential impacts on the embankment stability. These potential failure modes could be controlled through constraints on species, density of planting and adequate depth of soil above the structural embankment.

For large earthfill dams, an FMI process would identify several key risks relating to the body of the embankment and the behaviour of the foundation. Landscaping could obscure defects, so potential mitigation could include engineered sections (without landscaping fill) at critical sections which would allow for visual surveillance. Including suitable and comparable instrumentation in both engineered and non-engineered sections would allow for the behavioural trends of the dam to be understood and identification of any defects long before they could progress toward potential failure. Examples of key threats relating to the body of a dam and their foundations that may be identified as part of an FMI process are summarised in Table 1 below.

Methodology for the use of Failure Mode Identification in the design of new reservoirs

Table 1. Example FMI and Design Mitigations for Body of the Dam and Foundation

Threat	Progression	Design Mitigation
Body of dam	Degradation of embankment leading to loss of strength, increased saturation progressing to a stability failure	Wide core with protection to the top to reduce desiccation risks. Wide crest, shallow shoulders. Internal embankment inclined filter drainage
	Horizontal displacement leading to lateral spreading, cracking, settlement, loss of freeboard and overtopping.	Post-construction settlement allowance. Realtime monitoring of seepage, pore pressures and displacement.
	Risk of expansive material in the dam fill (e.g. pyrite)	Adequate Emergency Drawdown (EDD) provisions.
	Collapse compression due to high air voids (e.g. due to use of stiff fill such as over-consolidated mudstone)	Adequate surveillance, 'engineered' sections with no landscaping fill.
Foundation	Degradation of embankment from aging or foundation leading to loss of material strength or increased porosity leading to increased saturation progressing to a stability failure	Allowance for future raising. Trial embankment Unsuitable material (e.g. peat) removed from footprint Compaction of shoulder fill to achieve <5% air voids No expansive material near critical structures (culverts)
	Slip failure within foundation due to activation of relic shears (e.g. due to periglacial conditions)	GI to minimise risk with large OTPs, Trial embankment and compaction trials prior to construction. Slope stability to consider reasonable worst case with mitigation measures. Constraints on build rates relating to PWP response.
	Base heave in borrow pit	Appropriate assessment to consider realistic scenario.
	Differential settlement of tower and culvert leads to failure of tower and loss of water through culvert.	Foundation considers floatation. Flexible expansion joints, filter collar and settlement allowance Flexible tower top culvert joint Bulkhead culvert door considered

Interfaces between earthworks and structures, such as culverts, present significant risks due to the possibility of differential settlement causing hydraulic fracture. Identification and management of these potential failure modes can consider the construction methodology. A tunnelling method for example could be carried out after embankment construction when the initial consolidation has occurred but, depending on the size and length of the conduit, may be prohibitively costly and would make installing filter collars more challenging. An alternative cut-and-cover method would need to be constructed prior to the embankment so subject to a greater degree of settlement. Mitigation could then include use of a D-shaped culvert with sloping sides and a low-friction membrane to reduce the risk of cracking from differential settlement or zones of high stress. Installation of a filter collar would be straight forward but would need to be installed with a camber and include joints to allow for articulation to accommodate the expected settlement.

In the case of non-impounding reservoirs, inlet structures are also required. Potential failure modes relating to these structures would depend on the specific form they take. Consideration should be given to whether these structures would introduce additional low level penetrations or whether these could pass over the crest with double isolation, to mitigate the potential internal erosion risks.

Construction quality is critical, with inadequate supervision posing a major risk across multiple failure modes.

Design Assumptions affecting Reservoir Safety

The FMI process can provide the initial framework for how safety critical decisions could be governed, communicated, and assured through both the design and construction phases. To ensure the process is appropriately captured and communicated to clients, any future designers and key stakeholders, it is recommended to transform the output into a clear set of 'Design Assumptions affecting Reservoir Safety'.

A set of key design assumptions can be defined for specific 'assets' (e.g. embankment, inlet, spillway) and further broken down into each 'element' (e.g. upstream face, core), stating the design assumption and the associated failure mode(s) that each principle mitigates. This output can be specific to a snapshot of the actual design arrangement at the related stage of the projects development with the intention of communicating the key reservoir safety design elements and how these have been currently embedded into the design. However, the high level of detail could be too inflexible for

Methodology for the use of Failure Mode Identification in the design of new reservoirs

embedding into the planning and reference design but can be developed as the project progresses.

The design assumptions affecting reservoir safety could be developed by the Construction Engineer and Reservoir Safety team into a set of Mandatory Reservoir Safety Design Principles. These would be less prescriptive such that they can be embedded in the planning application and reference design whilst still allowing flexibility in compliance as the design develops.

An example of possible Mandatory Reservoir Safety Design Principles is presented as follows:

- Measures to prevent internal erosion without geotextiles and pore water pressures causing instability and considering geochemistry reactions and impacts;
- Designed to safely contain the design and safety check water balance conditions without overflowing;
- Dimension, profile and material specification inherently safe against hydraulic fracture, instability and weathering;
- Landscaping not to interfere with reservoir safety features;
- Monitoring and surveillance strategy based on critical failure modes;
- Buried structures to incorporate features to prevent contact erosion;
- Compliant drawdown with unrestricted testing and no unauthorised operation.

These could continue to be developed as projects progress with the intention for them to be ultimately bound into the procurement of the works.

CONCLUSION

The FMI process is a critical part of RARS which was developed primarily for application to existing reservoirs. However, the approach can be adopted and applied to the design of new reservoirs at an early development phase.

The methodology described in this paper follows the RARS guide by identifying the threats (a condition which may initiate a hazard occurring) and the progression (failure mode which is the mechanism by which a hazard could lead to a failure of the dam). Following this stage, rather than assess whether the risks would be credible or significant, the focus can be on determining physical solutions to be considered for incorporation with the intention of designing out or mitigating the failure modes.

The FMI process should remain live throughout the design of the project and be continually updated and reviewed as the design develops or new data becomes available. It should be developed and progressed in collaboration with the Construction Engineer, Client and dam specialists, including geotechnical engineers and key stakeholders.

The FMI process provides a framework to identify and mitigate reservoir safety risks early in the design. It has also provided a detailed and robust justification for mandatory reservoir safety design principles that can be embedded at planning stage and eventually the procurement and construction of the schemes.

REFERENCES

- Brown A J (2025), Risk-based or risk-informed reservoir safety management? Reflections on current practice. *Dams and Reservoirs* **35(2)** 51–67.
- EA (2013) (erratum 2016) *Guide to Risk Assessment for Reservoir Safety Management (RARS)*– SC090001/R1. Environment Agency, Bristol, UK
- ICE (2015) *Floods and Reservoir Safety. 4th edition. (FRS4)*. Thomas Telford, London, UK.



The British Dam Society

Assessing the impact of the UK's glacial legacy on the design of dams – past, present and future

M WEBB, Mott MacDonald

H FITZGIBBON, Mott MacDonald

M MULHOLLAND, Mott MacDonald

SYNOPSIS

The UK is embarking on a once-in-a-generation reservoir construction drive. These nationally significant infrastructure projects aim to secure water resilience amid population growth and a changing climate. No major reservoir has entered supply in the UK since Carsington in 1992 – the dam which suffered geotechnical failure, in part, due to its glacial legacy.

As part of the early design development for Anglian Water's two flagship reservoirs in the east of England, Fens Reservoir and Lincolnshire Reservoir, the potential impact of the glacial legacy has been investigated.

Given the geological and glacial history of the UK, relic geohazards are likely. Notably, pre-existing shear surfaces, on which displacement will have occurred and strength is typically close to (or at) residual strength. The persistence, frequency, orientation, roughness and surface asperities of shears ultimately control the foundation strength of strata and thus design (specifically slope stability).

Large Observational Trial Pits (OTPs), as described in the ICE Specification for ground investigation 3rd edition (AGS, 2022) are seldom used in modern investigations; however, they can enable safe entry and forensic description of the soils that traditional Ground Investigation techniques cannot. A series of OTPs were undertaken at the Lincolnshire site in 2025. The findings from those completed have proved invaluable in beginning to understand the periglacial and glacial processes and subsequent hazards to inform a robust and resilient design.

This paper will set out the importance of a robust desk study to inform a targeted investigation to ensure the correct level of geotechnical understanding is available during early project development. It will focus on the Fens Reservoir as an example of the desk study process. It will not discuss the interpretation of the field observations, as that will be the subject of a future publication.

Assessing the impact of the UK's glacial legacy on the design of dams – past, present and future

INTRODUCTION

The Fens Reservoir is one of several nationally significant schemes addressing the increasing deficits in the United Kingdom's (UK) public water supply. Promoted by Anglian Water and Cambridge Water, it is progressing toward Development Consent Order (DCO) submission under the Regulators' Alliance for Progressing Infrastructure Development (RAPID) framework. Once complete, it will be one of the first reservoirs to be built in the UK for water supply since Carsington Dam in 1992. The reservoir will strengthen the UK's water supply, enhancing resilience in the east of England to drought and flooding from a future of changing climate.

Following Glossop (1968) "*... if you do not know what you are looking for, you are not likely to find much of value*" – a detailed geological desk study assessing the impact of glacial legacy on the Fens Reservoir was undertaken as part of the early design stage. Glossop's principal underpinned the desk study process, highlighting their importance and the considerations in defining a targeted and specific ground investigation. This paper firstly introduces the glacial history of the UK, then outlines the key stages of the desk study for the Fens site before outlining the design of the OTPs.

Although the same process was undertaken for both Reservoirs, this paper focuses solely on the Fens Reservoir.

GLACIAL HISTORY OF THE UK

The Quaternary (2.6 Ma to present) comprises the Pleistocene (~2.5 Ma to 11,700 years ago), and the Holocene (11,700 years ago to present) epochs (Lukas et al., 2017). The Pleistocene was an interval of rapidly changing climate conditions and saw the intensification in the frequency and magnitude of high- to mid-latitude glaciation (Berry et al., 2020). Within the last ~500 Ka of the Pleistocene, several glacial-interglacial (GIG) cycles affected the UK, with the most prevalent to ground engineering briefly summarised below (Berry et al., 2020):

- The Anglian glaciation (~460 Ka) saw ice sheets reach as far south as London and Bristol, with periglacial conditions further south.
- The Tottenhill (also referenced as Wolstonian) Glaciation (~160 Ka), where much of the UK experienced periglacial conditions.
- The Devensian glaciation (~27 Ka), ice sheets advanced to north Norfolk and the Midlands, and an intense periglacial climate was experienced beyond the ice sheets in the south of the UK.
- The Loch Lomond Readvance (~13 Ka), which was a transient period of localised glacial and periglacial climates at mid- to high-latitudes.

- The Little Ice Age (16th to 19th centuries) included periodic freezing of the River Thames. Glaciers did not return, but freeze-thaw periods likely intensified across lowland areas of the UK.

The GIG cycles of the Pleistocene had profound implications for the geomorphology of the UK, notably creating a series of geohazards which remain prevalent to the present day. During cold periods, ice sheets expanded, with prevailing glacial environments creating characteristic landforms through the erosion and deposition of glacial sediments. Areas beyond the limits of glaciers experienced periglacial conditions, resulting in widespread ground freezing and development of ground ice. This is particularly prominent in clay and silt-rich soils where pore spaces promote capillary flow and the development of ice (Berry et al., 2020).

Cyclical freeze-thaw processes cause porewater pressures variations that alter effective stress, and subsequently geotechnical properties of the soils. If these effects go unnoticed, they can present significant geohazards to engineering in relict periglacial landscapes.

DESK STUDY PROCESS

The geotechnical failure of Carsington Dam during construction in 1984 highlighted the need for a robust understanding of the potential impact of the UK's glacial legacy on reservoir and dam design. As highlighted by Fookes (1997) and required by UK codes, a detailed geological desk study is an essential step for all engineering projects to manage ground risk. A focused desk study was undertaken for the site to identify the potential risk of relict glacial and periglacial geohazards which may affect the reservoir design and construction. The following sections outlines the key steps undertaken within this desk study process.

Regional Geological History of the Fens

The first step in the desk study process was understanding the documented regional geological history of the site. This is briefly summarised below.

Bedrock Geology

The site is underlain by the Ampthill Clay and West Walton Formation (undifferentiated), and subsequently the Oxford Clay Formation, of the Ancholme Group. The bedrock clays were deposited within warm shallow seas of the Jurassic ~170 Ma. Later deposition of Cretaceous and Eocene strata occurred between the Jurassic and the Pleistocene and was later eroded.

Assessing the impact of the UK's glacial legacy on the design of dams – past, present and future

Quaternary geology

The rapidly changing climate conditions and intense glacial-interglacial cycles of the Pleistocene (as described above) significantly transformed the geology of the Fens Reservoir site. The desk study identified three key GIG cycles which were likely to have impacted the site: namely the Anglian, the Tottenhill, and the Devensian glaciation. The location of the Fens Reservoir site with respect to these are schematically represented in Figure 1.

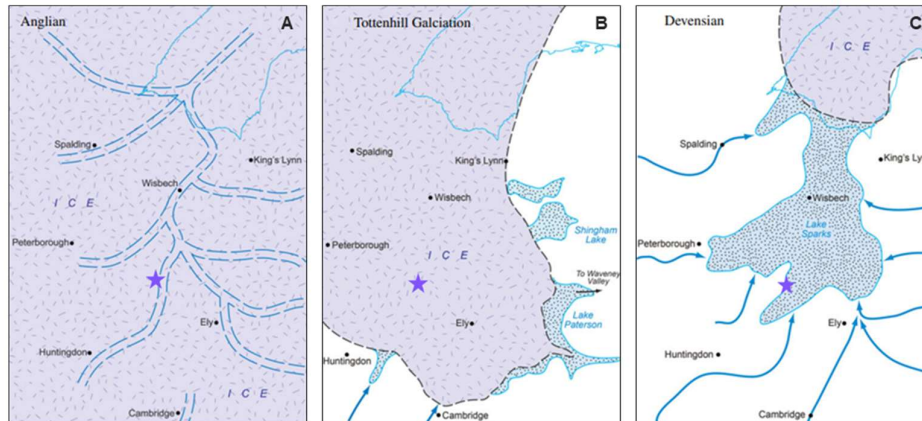


Figure 1: The location of the proposed Fens Reservoir site (starred) with respect to the Anglian, the Tottenhill, and the Devensian Glacial extents. Adapted from Gibbard et al. (2018).

Considering the position of the Fens Reservoir site with respect to the various glacial extents in Figure 1 above, the Jurassic Clays of the Ancholme Group are likely to have been affected by both glacial and periglacial conditions, seeing repeated growth and degradation of ground ice throughout the Quaternary period. This identified the geotechnical risk that repeated freezing and thawing of the ground had impacted the site, and further investigation of this was therefore critical to reservoir design, as well as understanding the likelihood of encountering relict geohazards.

Form of Relict geohazards

Once it was identified that the glacial legacy of the site meant there was a risk of encountering geohazards, the potential form these may take was considered. Pre-existing shear planes can form in various geological settings where rocks and soils have experienced differential stresses, with each type of pre-existing shear plane exhibiting its own unique characteristics depending on the conditions and forces present in their environment. However, ultimately, they all represent pre-existing planes of weakness within material which need to be accounted for, regardless of scale or formation process, prior to the commencement of engineering works.

The primary geotechnical concern with shear surfaces is the reduced shear strength due to particle reorientation along the slip surface during historical displacement, which will directly impact the strength value utilised in design.

There are three main types of pre-existing shear planes: Tectonic shear planes, Glaciotectonic shear planes and Periglacial shear planes. The potential for all three pre-existing shear plane types was considered in the geological desk study for the Fens Reservoir site; however, it was determined that the most probable, given the glacial history of the site, were glaciotectonic and periglacial shear planes. Although the chance of encountering tectonic shear planes cannot be ruled out, they are not discussed further in this paper.

Glaciotectonic shear planes

Glaciotectonism refers to the structural deformation of sediment or bedrock caused by glacial stresses (Van der Wateren, 2002). It can occur in either brittle or ductile regimes, forming glaciotectonic shear planes that accommodate deformation or enable movement. Figure 5(A) presents a conceptual model illustrating this.

Periglacial shear planes

Cyclical freeze-thaw during periglacial conditions causes variations in pore-water pressures, often driving slow downslope movement of soil (solifluction) (Berry et al., 2020). These processes can cause shallow translational failures of near surface soils, resulting in polished, often striated, shear surfaces along which material has moved. Periglacial shear planes have been observed on slopes as slack as $1.5\text{-}2^\circ$ (Skempton and Weeks, 1976). Shear planes may also form through other mechanisms, such as thaw collapse. Spink (1991) categorised seven types of periglacial shear, all of which could be present at the Fens site.

Evidence of Glacial Legacy

Following identification that the site has been affected by both glacial and periglacial conditions, evidence for these was reviewed. This included a site walkover (not discussed further in this paper), satellite imagery, LiDAR data, and existing ground investigation data to constrain the impact of the site's glacial legacy and determine whether a more targeted ground investigation was required. The key findings are summarised below.

Assessing the impact of the UK's glacial legacy on the design of dams – past, present and future

Satellite imagery

Compelling evidence for ground ice formation within the bedrock soils at the Fens Reservoir site was identified, with aerial imagery identifying probable ice wedge polygons. These form due to ice generation, under periglacial conditions where ground cooling is sufficient to overcome the tensile strength of the clay, causing cracks. As groundwater enters the cracks and refreezes, this process is intensified by the expansion of groundwater. Ice wedge polygons can disturb ground structure and influence both strength and permeability. An example is shown in Figure 2.



Figure 2: Ice wedge polygons within the Fens site (GR: 541175E, 290567N). Source: Esri, Intermap. Maxar (Vivid) imagery captured 11 July 2022.

LiDAR Data

The next dataset reviewed was composite DTM LiDAR data (1m resolution) from Defra (2022), shown in Figure 3. While higher-resolution, site-specific survey data would allow a more detailed assessment, this was not available at the time of the desk study.

The LiDAR data indicates a generally flat, subdued, topography, with slopes rarely exceeding 2-3°, Figure 3(B). A hillshade of the LiDAR data in Figure 3(A) reveals a series of east-west trending, dendritic roddons which are raised above the surface. However, these are paleo-intertidal drainage channels which were active during the Holocene, and are not directly related to glacial or periglacial conditions (but a product of the release of ice locked water during glacial retreat); but are likely in part to have masked the previous topography that had been directly influenced by glacial and periglacial conditions. The hillshade also shows more elevated topography to the

northwest of the site, corresponding to steeper slope angles, and exposure of the bedrock clays.

The data shows no clear evidence for large-scale periglacial processes, such as landsliding or significant slope movements. However, the absence of steep slopes does not preclude the presence of pre-existing shear surfaces, as previously mentioned.

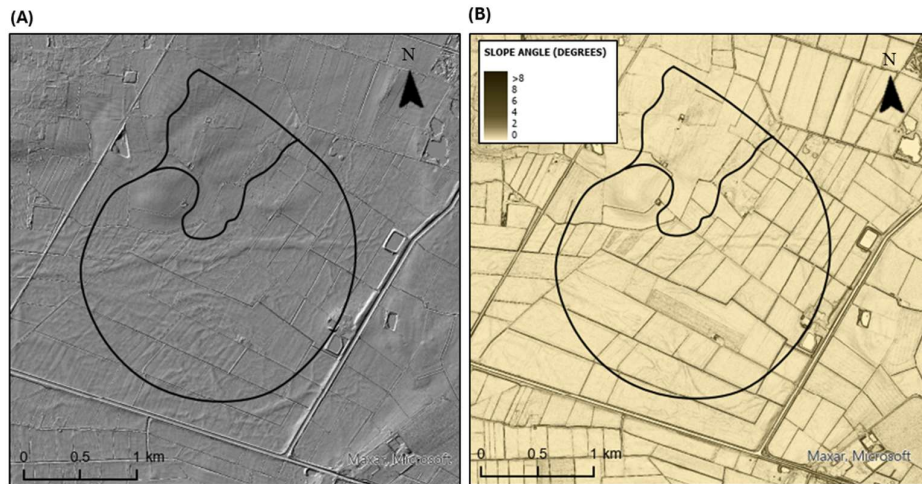


Figure 3: LiDAR data for the Fens Reservoir site displaying A) Hillshade, and B) Slope Angle. Source: LiDAR data from Defra (2022) imported into ArcGIS Pro (2025).

Site-Specific Ground Investigation Data

The next stage is to assess the findings of any available ground investigation (GI) data available for the site. At the Fens Reservoir site, two phases of GI (2023 and 2024) had been completed when undertaking the detailed desk study. Traditional GI techniques are poorly suited to identifying pre-existing shear surfaces, as boreholes provide discrete samples unlikely to capture the discontinuous and highly variable nature of pre-existing shear surfaces. Nevertheless, both phases of GI, recorded occasional “polished” fissures, indicating possible movement, along with evidence of glacial reworking and weathering of the bedrock clays. As an example, Figure 4 presents observations noted from a trial pit.

In (A), a slickensided siltstone band within the bedrock clays (2.5mbgl) indicates possible glaciotectionic deformation, with lineations potentially the result of shallow thrusting of material beneath an ice sheet. As siltstones are more resistant to glacial reworking than clay, the slickensides may reflect scouring of the bedrock as an ice sheet is thrust over more resistant material. In (B), bedrock clays (3.0mbgl) have been significantly reworked, forming randomly oriented, undulous, relatively clean surfaces.

Assessing the impact of the UK's glacial legacy on the design of dams – past, present and future

Additionally, evidence of weathering within the bedrock was identified to a maximum depth of ~7.5mbgl. This paper does not describe in detail the weathering observed, and evidence of this. However, this was a key element in the desk study process in order to design suitable further investigation.

The findings of the remaining ground investigation are beyond the scope of this paper. However, it did confirm the likely influence of both periglacial and glacial climates at the reservoir site and highlighted the difficulty in identifying glacial or periglacial features with conventional ground investigation techniques, and therefore that a focused and more specific investigation was required.

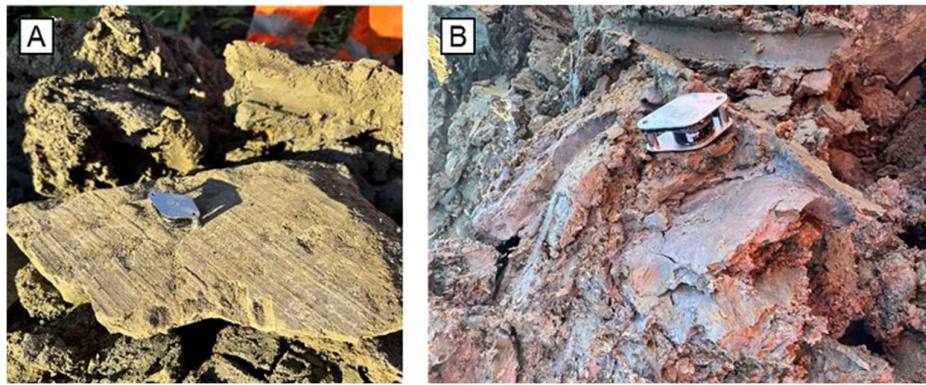


Figure 4: Observations from a trial pit (GR: 542181E, 289994N) undertaken at the Fens Reservoir site. Source: MM GI Site Supervisors.

Development of Conceptual Model

Based on the information collated from the above-described steps, a series of conceptual models was produced to summarise the potential ground conditions and relict geohazards associated with the GIG cycles affecting the Fens Reservoir site. Examples are provided in Figure 5. In (A), the Tottenhill Glaciation, where the Fens Reservoir was directly glaciated, in (B), when the Fens Reservoir site was exposed to periglacial conditions (as shown earlier in Figure 1), and (C), the present day, displaying the dendritic roddons, ice wedge polygons, and influx of tidal flat deposits following several marine transgressions (yellow).

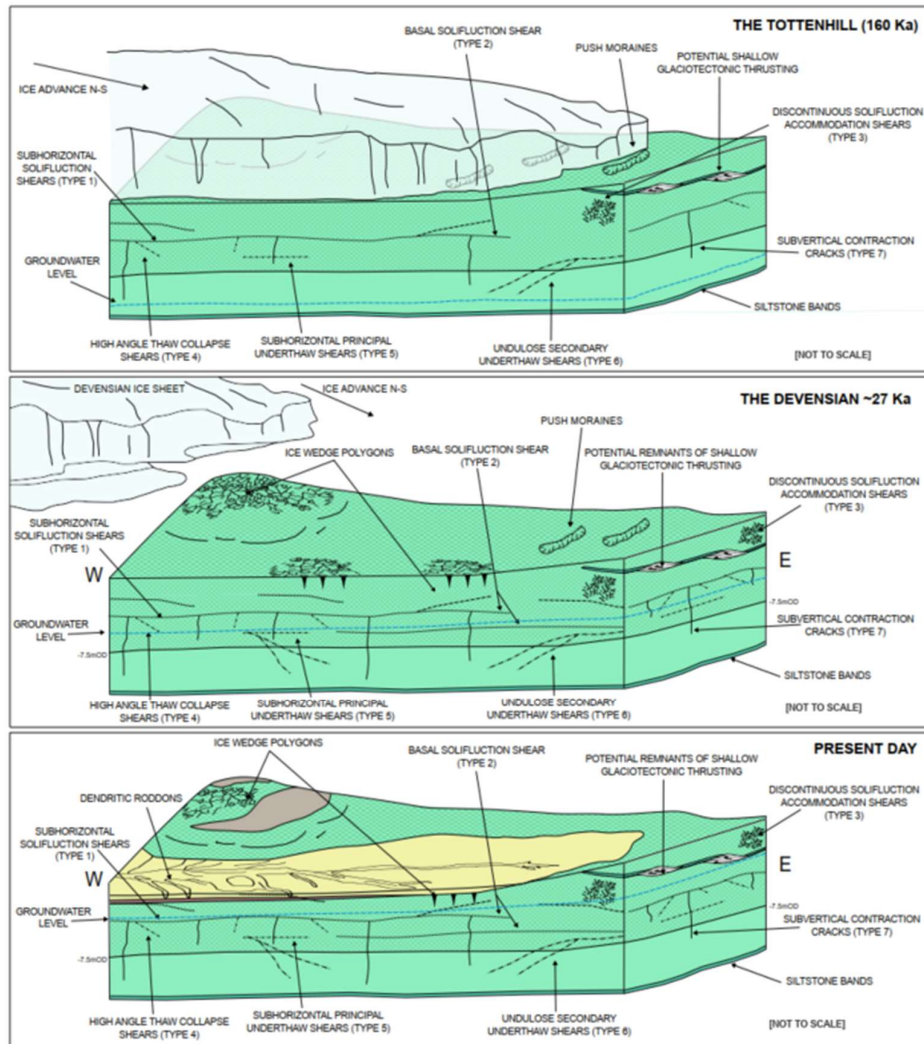


Figure 5: Conceptual model of the Fens Reservoir site: A) the Tottenhill Glaciation, B) the Devensian, C) Present day. Note this model is not to scale and should not be relied upon as a conclusive or definite representation of the conditions present at the site. Source: MM Internal (hand-illustrated).

OBSERVATIONAL TRIAL PIT (OTP) DESIGN

Following completion of the desk study, a focused ground investigation to better understand the presence (or absence) of periglacial and glacial features was designed. Trial Pits allowing personnel entry, as described in the ICE Specification for Ground Investigation 3rd edition (AGS, 2022) are seldom used in modern investigation, enable safe entry and forensic description of soils that traditional GI techniques cannot. These were deemed to be the most effective method to investigate the glacial legacy of the site.

Assessing the impact of the UK's glacial legacy on the design of dams – past, present and future

The primary aim of the OTPs was to maximise opportunities to visually identify, map and record discontinuities, and to describe the geological strata to confirm or, negate the presence, persistence and properties of pre-existing shears within the bedrock. This requires large, exposed soil faces and a period of time to allow surface soils to dry out.

These requirements, together with the desk study findings, informed the OTP design. Additional factors, including constructability, temporary works, health and safety, groundwater (and surface water) management, landowner impacts, planning constraints, programme, ecology and archaeology were also key. Although not discussed in detail here, they must be considered early, as they are as influential as the technical aspects.

The OTP geometry was designed to investigate bedrock zones most likely affected by glacial / periglacial processes. As outlined by Spink and Norbury (1993), understanding weathering profiles in bedrock clays is key to identifying horizons where such features are likely encountered. A minimum excavation depth of 6m was selected to adequately observe the weathering profile; this was constrained by the considerations noted above and represents a compromise. An example in progress OTP is shown in Figure 6



below.

Figure 6: OTP at the Lincolnshire Reservoir site 2025, Blackwell Earthmoving Limited and Anglian Water.

Another key consideration was the drying period of excavation surfaces, which was required to enable easier identification and description of shear surfaces. The required duration is influenced by various factors such as the in-situ water content of the material, the weather conditions prior to, during and after excavation and the orientation of the pits. Regular inspections are therefore essential to determine the optimum point at which to log the OTPs. Over drying can hinder identification of features and may necessitate further excavation.

High quality, detailed logging of the OTPs is essential to maximise data capture. As a minimum, the following should be recorded: detailed sketches of logged areas (including strata horizons, discontinuities, inclusions), a full geotechnical description of each geological horizon and a factual description

of the degree and extent of weathering. Each discontinuity (or set) should be described in detail, including type, strike, dip, weathering, spacing, persistence, roughness, surface condition, aperture, block size, infill, mineral coatings and/or associations and waviness where applicable.

Consideration is needed to determine the most appropriate logging approach (e.g. top down, bottom up etc) to meet technical requirements. Health and safety must also be considered, as logging may constitute working at height if not undertaken from the OTP base. Temporary works to enable alternative logging methods (e.g. access to multiple benches) should be considered. The desk study helps identify likely shear types and horizons, which ultimately may influence the optimum logging methodology.

Acquisition of suitable geotechnical samples is key. Alongside conventional samples, block samples should be obtained where possible, as OTPs offer a rare chance to acquire high quality samples. Where feasible, sampling along identified shear surfaces allows testing of the mobilised strength of a slip surface, a critical parameter that will ultimately influence reservoir design.

Using drones (or similar) to record feature locations, alongside high-quality digital photography, significantly enhances the value of the data gathered.

Given the cost and planning required for such investigations, opportunities to undertake additional tests should be considered. These could include, for example, undertaking a compaction trial or a geochemical weathering trial.

CONCLUSIONS

A robust desk study is essential to identify potential geohazards associated with a site's glacial legacy and to inform appropriate investigation strategies. If pre-existing shear surfaces are found to be pervasive and persistent, the design may need to consider a strength value at, or close to, residual strength. This would significantly impact on the stability of the proposed embankments. Design, cost, programme and environmental impacts would follow. Early understanding of glacial legacy is therefore critical to avoid failures such as Carsington and to deliver resilient reservoir infrastructure.

The process set out within this paper was followed and led to the successful completion of OTPs at the Lincolnshire Reservoir site in 2025. The findings, and interpretation of these, are the subject of a future publication.

ACKNOWLEDGEMENT

The authors would like to thank Anglian Water and Cambridge Water for permission to publish this paper, and to colleagues in Mott MacDonald for guidance and support.

Assessing the impact of the UK's glacial legacy on the design of dams – past, present and future

REFERENCES

- AGS (2022). *UK Specification for Ground Investigation, 3rd Edition*. ICE Publishing.
- Glossop R (1968). The rise of geotechnology and its influence on engineering practice. *Geotechnique*, Vol 18, pp. 107-150
- Lukas S et al. (2017). The Quaternary. Engineering Geology and Geomorphology of Glaciated and Periglaciated Terrains; *Engineering Group Working Party Report*, Vol. 28, pp. 31-57.
- Berry T et al. (2020). Periglacial Geohazards in the UK. Geological Hazards in the UK: Their Occurrence, Monitoring and Mitigation; *Engineering Group Working Party Report*, vol. 29, pp. 259-289.
- Fookes P G (1997). Geology for Engineers: The Geological Model, Prediction, and Performance. *Quarterly Journal of Engineering Geology*, Vol. 30, pp. 293-424.
- Gibbard P et al. (2018). Pleistocene glaciation of Fenland, England, and its implications for the evolution of the region. *Royal Society Open Science*, Vol. 5.
- Van der Wateren D (2002). Processes of Glaciotectonism. *Modern and Past Glacial Environment*, pp. 417-443.
- Skempton A and Weeks A (1976). The Quaternary History of the Lower Greensand Escarpment and Weald Clay Vale near Sevenoaks, Kent [and discussion]. A Discussion on Valley Slopes and Cliffs in Southern England: *Morphology, Mechanics and Quaternary History*, vol. 283, no. 1315, pp. 493-526.
- Spink T W (1991). Periglacial discontinuities in Eocene clays near Denham, Buckinghamshire. *Quaternary Engineering Geology; Engineering Geology Special Publication*, Vol. 7, pp.389-396.
- Defra (Department for Environment Food and Rural Affairs), “*Defra Survey Data Download*,” (2022). <https://environment.data.gov.uk/survey>.
- Spink T W and Norbury D R (1993). The engineering geological description of weak rocks and overconsolidated soils. *The Engineering Geology of Weak Rock; Engineering Geology Special Publication*, No. 8, pp. 289-301.



Geochemistry considerations for the design of new earthfill embankment dams in the UK

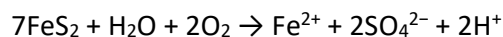
T W ST JOHN, Mott MacDonald

SYNOPSIS

Several proposed UK reservoirs are located on principal sulfate and sulfide-bearing strata. Exposing pyritic materials as part of the multi-million-cubic-metres earthworks schemes will induce geochemical reactions. The implications of oxidation of sulfides in fills and foundations include changes in material volume, shear strength and permeability. Carbon dioxide gas may be produced from the formation of sulfates whilst mobilised sulfates and acidity can attack concrete and clog filters. Those processes can contribute to adverse performance of safety-critical elements and pose operational and environmental challenges. This paper describes geochemistry hazard characterisation at the new Lincolnshire Reservoir site, as part of the ground risk management strategy. The role of investigation including large observational trial pits and petrographic testing in the development of a conceptual model is discussed. The proposals for accelerated weathering tests and a geochemical trial embankment are described to illustrate how those innovations can aid understanding of material changes and enable appropriate embankment design and material specification, selection and treatment.

INTRODUCTION

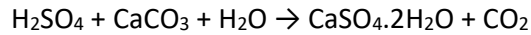
The majority of the UK major reservoirs in design development or construction are located on principal sulfate and/or sulfide-bearing strata (Table 1). Many of the schemes will require use of locally-won sulfur-rich clays to form the embankments, whilst associated infrastructure will be founded in or on those clays. On atmospheric exposure of the clays, sulfides begin to oxidise and the extent and rate of oxidation is a function of mineralogy, temperature, pH, organic content, permeability and bacteria. Pyrite oxidation is summarised as:



pyrite + water + oxygen → ferrous iron + sulfate + hydrogen ions (acidity)

Oxidation of Fe^{2+} produces orange-brown iron-rich precipitates (ochre) and releases further acidity. SO_4^{2-} can react with calcium carbonate in the host material to form gypsum ($\text{CaSO}_4 \cdot 2\text{H}_2\text{O}$) and carbon dioxide gas:

Geochemistry considerations for the design of new earthfill embankment dams in the UK



Sulfuric acid + calcite + water $\rightarrow$ gypsum + carbon dioxide

In the presence of concrete, SO_4^{2-} can react with cement phases to produce the sulfate minerals ettringite and thaumasite. Growth of new sulfates is associated with a net volume increase which may be 50-500% (Penner et al, 1973; Fasiska et al, 1974; Cripps and Edwards, 1997). Growth likely commences in pores and discontinuities and may be followed by material densification and finally displacement. Growth can continue until confining stress exceeds crystallisation pressures in the range 300-600kPa (Hawkins, 2014). Some strata contain 'primary' sulfates (e.g., evaporite deposits), as distinct from those formed by post-depositional oxidation of sulfides.

Excavation, stockpiling, conditioning and groundwater regime changes required for reservoir construction can induce rapid sulfide oxidation. Those processes influence the dissolution of sulfates into groundwater as well as transport pathways for SO_4^{2-} to sites of new sulfate mineral growth. Adverse implications for civil engineering include i) fill expansion, ii) mineral leaching, iii) strength, grading and permeability changes, iv) filter clogging, v) sulfate attack of concrete and metals, vi) low pH, high sulfate, high iron drainage and vii) CO_2 gas generation. With many new reservoirs in design, it is essential that lessons from cases of complex geochemistry problems are reviewed and that mitigation is implemented to prevent recurrence.

Table 1. Proposed UK reservoirs^a on principal^b sulfide/sulfate strata foundations

Reservoir name	Region	Bedrock foundation
Lincolnshire	Sleaford, Lincolnshire	Oxford Clay Formation (J)
Fens	Chatteris, Cambridge	Ampthill Clay and West Walton Formation (Undifferentiated) ^c (J)
Broad Oak	Canterbury, Kent	London Clay Formation (E)
White Horse	Abingdon, Oxfordshire	Kimmeridge Clay Formation (J)
Havant Thicket	Havant, Hampshire	London Clay Formation (E)
Cheddar II	Cheddar, Somerset	Mercia Mudstone Group (T)

^a Other reservoirs located on principal sulfide/sulfate bearing strata may be in very early-stage feasibility assessment but are not included here.

^b As defined by BRE Special Digest 1 (BRE, 2005).

^c Interpretation to date indicates that the West Walton Formation underlies site.

J = Jurassic mudstones that weather to clay

E = Eocene clay

T = Triassic mudstones and siltstones with thin beds of gypsum, that weather to clays and silts.

Sulfur geochemistry legacy at UK earthfill dam reservoir sites

The embankment at Carsington was formed of pyritic mudstone. During investigations into the embankment failure (not caused by geochemical processes), limestone drainage blankets were found clogged with secondary minerals whilst buried concrete had suffered sulfate attack (Reid et al, 2005). Reconstruction aimed to minimise pyrite oxidation, with allowances made for future material degradation (Chalmers et al, 1993; Macdonald and Reid, 1991). Materials were zoned, with completely weathered material used for the core and slightly/moderately weathered material used for the shoulders. Siliceous gravel was used for the replaced drainage and the basal drain is maintained in a saturated state. A gravel capillary break was placed on the downstream shoulder crest. Between 1991 and 1994 the average annual fill weight lost owing to geochemical processes was estimated as 0.00017% (or 8,335kg and 2.62m³) of the total fill (Reid et al, 2005). Deposition of ochre at emergence of the seepage remains a maintenance concern and shows continued sulfide oxidation 30-35yrs post-construction.

The embankment at Roadford was formed of sandstone and mudstone, with 1.1% pyrite on average (Reid et al, 2005, Hopkins et al, 2010). Drainage from the trial embankment was acidic, with pH <4. Design mitigation included an allowance for material strength degradation, covering the upstream embankment face with a membrane to reduce air and water ingress, and treatment of drainage to precipitate iron. Between 1990 and 1991 the fill weight lost owing to geochemical processes was estimated as 0.0039% (or 82,570kg and 25.15m³) of the total fill (Reid et al, 2005).

The proposed Lincolnshire Reservoir

Design of the 55Mm³ capacity reservoir is being progressed to Development Consent Order via milestone 'Gates'. As part of Gate 3, Mott MacDonald was responsible for geotechnical design of the reservoir site for the promotor, Anglian Water, including the ground risk management strategy. The reservoir site is located about 7km southeast of Sleaford and comprises subtle east-west trending ridges and troughs which drain east to the Lincolnshire Fens. The ground conditions are of Glacial Till overlying bedrock of the Oxford Clay Formation, with the thickness of Till varying across the site and locally absent (Figure 1). As of March 2026, the design was full-perimeter clay embankment of height 1.5-18m and c. 9km length. Structural embankment slopes are anticipated to be 1v:6.5h or shallower, overlain by much shallower landscaping. A 'chimney' drain will abut the downstream side of the clay core and discharge to a drainage blanket. The many millions of cubic metres of fill for the earthworks are to be sourced from within the reservoir footprint whilst drainage materials are anticipated to be imported.

Geochemistry considerations for the design of new earthfill embankment dams in the UK

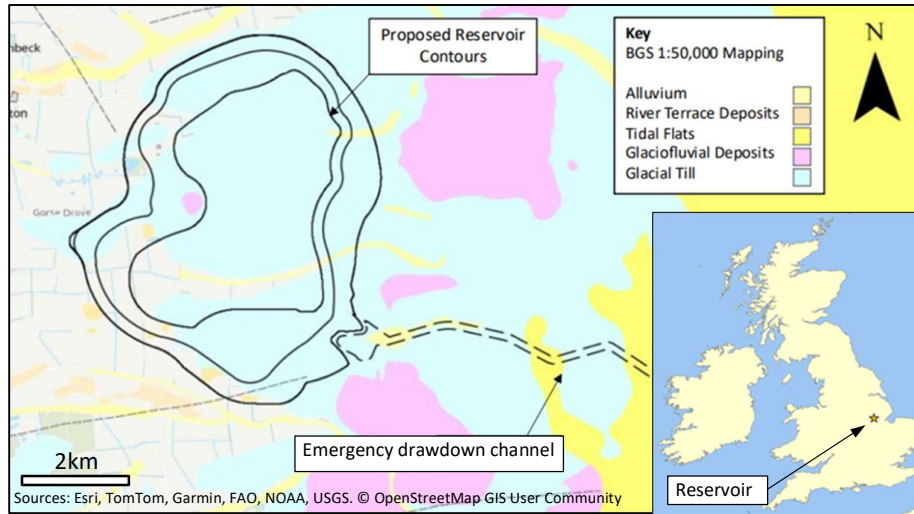


Figure 1. Reservoir site and superficial geology. Bedrock of the Oxford Clay Formation underlies the whole reservoir site.

SULFUR GEOCHEMISTRY OF THE LINCOLNSHIRE RESERVOIR SITE

Assessment of the site sulfur geochemistry followed best practice presented in BRE SD1 (2005). The Oxford Clay is known for its high sulfur content and there is a history of engineering problems associated with its sulfur chemistry (Hawkins, 2014; St John, 2020). The chalky glacial tills of eastern England are often sulfur-rich and lime stabilisation of such material has been associated with highway pavement distress (Hawkins, 2014). Historical boreholes in the region confirm the presence of sulfur phases including selenite (gypsum) crystals within 3.5m of the ground surface.

Phase 1 ground investigation: conventional approaches

Initial investigation was undertaken in 2024 to define the ground model, identify geohazards and determine preliminary geotechnical parameters. With respect to geochemistry, it aimed to i) characterise groundwater mobility, ii) identify sulfur-species and their distribution, iii) determine geochemical concentrations to define the Design Sulfate (DS) Class and iv) enable development of a conceptual geochemistry model. Further conventional investigation was undertaken as part of a Phase 2 investigation in 2025 however the results were not available at the time of writing.

The Glacial Till was described as stiff to very stiff orangish brown to dark brown and grey mottled fissured sandy gravelly silty CLAY, with the gravel including chalk and limestone. The Oxford Clay was firm to stiff brown-grey where weathered, becoming stiff to very stiff fissured grey to dark grey

St John

slightly sandy silty CLAY with rare fossils. Below 15-20mbgl, the Oxford Clay was extremely to very weak dark grey MUDSTONE.

Sulfur-species were identified in almost all exploratory holes. In the Glacial Till, gypsum was present as 'rare' to 'frequent' crystals up to 25mm and generally at 1-2.5mbgl. Pyrite was not identified. Within the Oxford Clay, gypsum was present as 'rare' to 'frequent' crystals up to 20mm and generally within 3-5mbgl, however 'rare' gypsum was recorded up to 19.5mbgl. Pyrite in the Oxford Clay was present as 'rare' to 'abundant' nodules up to 40mm, as fossil replacements and as sand sized crystals, generally below 7-11mbgl.

The pH, total sulfur, acid soluble sulfate and water soluble sulfate of soil samples were determined whilst pH and sulfate contents were determined for groundwater samples (Figures 2 and 3). Total potential sulfate, oxidisable sulfides, original pyrite content (EP_0) and current pyrite content (EP_C) were calculated to understand potential future sulfate from sulfide oxidation.

Testing showed that the Glacial Till is leached of sulfur in the upper 1-1.5mbgl. Water soluble sulfate, acid soluble sulfate and total potential sulfate peak around 2mbgl, reducing below that. Oxidisable sulfides values are low (indicative of oxidised pyrite) above 2mbgl, however below 2mbgl pyrite is likely present. On average, the Glacial Till is 50% oxidised and the EP_C is in the range 0.2-4.4% below 2mbgl, i.e., there is significant sulfide present which could be oxidised if exposed. Soil pH is alkaline, indicating that acidity generated by sulfide oxidation is buffered. Sulfate in groundwater ranges widely from 8-1900mg/l and at the upper bound is higher than determined from water-leaching of soil specimens, indicating possible groundwater equilibration with the underlying Oxford Clay.

Testing of the Oxford Clay showed that water soluble sulfate varies widely. Acid soluble sulfate peaks around 2mbgl (1-1.5m below the top of the unit), reducing below that. Oxidisable sulfides indicate consistent presence of pyrite, with an EP_C generally in the range 0.5-5%. On average, the Oxford Clay is 22% oxidised. Further testing is required to understand the distribution of sulfur >10m below the top of the unit. Similar to the Till, soil pH is alkaline. Groundwater pH was in the range 8-9.6, i.e., more alkaline than soil pH and suggestive of equilibrium with calcareous mudstones. Groundwater sulfate ranges widely from 7-2350mg/l.

X-ray diffraction (XRD) testing showed that the Oxford Clay mineralogy (median weight %) was calcite (24%), illite-mica (23%), quartz (18%) and kaolinite (13%). Interlayered illite-smectite was <10% and isolated smectite was not identified. Gypsum was largely "not quantified" or 0% gypsum was reported. Pyrite accounted for typically 2-4%; in agreement with the EP_C .

Geochemistry considerations for the design of new earthfill embankment dams in the UK

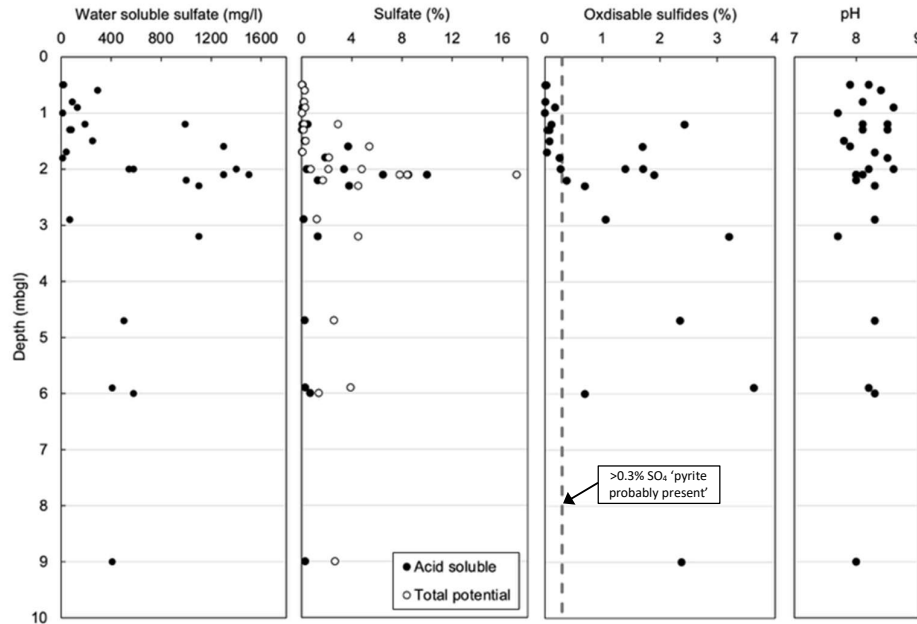


Figure 2. pH and sulfur geochemistry vertical distribution for Glacial Till.

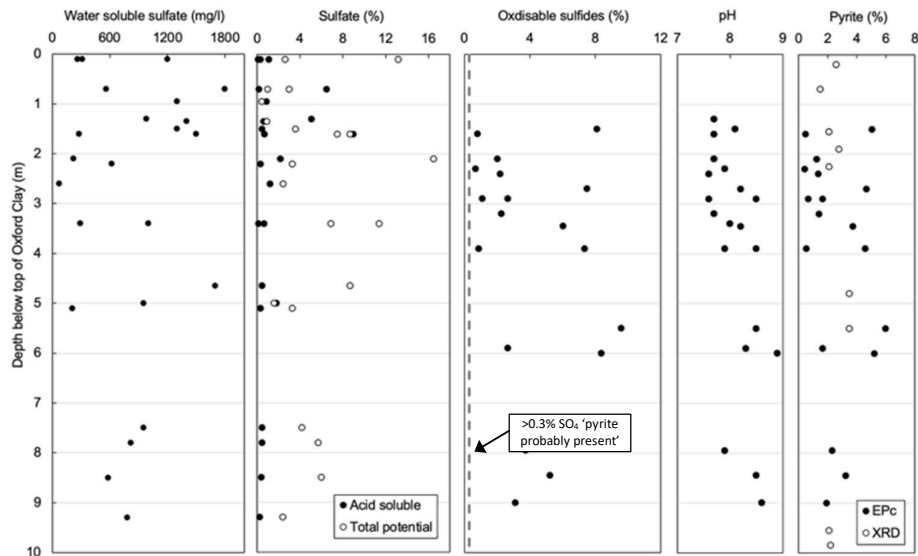


Figure 3. pH and sulfur geochemistry vertical distribution for Oxford Clay.

Phase 2 ground investigation: large observational trial pits

Observational trial pits were excavated within the reservoir footprint in 2025 (Figure 4). Typical plan dimensions were 25m x 25m, with a depth of around 5m. Access was made for person entry and logging. Results of geochemical tests on samples were not available for review at the time of writing. The pits offered an opportunity to document chemical weathering including the

vertical and lateral distribution of sulfur-species, at a scale typically only seen during construction of major earthworks schemes. Each pit was open for several weeks. Observations revealed pervasive evidence of geochemical weathering of the Oxford Clay and to a lesser extent the Glacial Till. Leaching of sulfur was apparent in the near-surface of both materials.



Figure 4. Example observational trial pit (Oxford Clay, (OxC) at surface).

Gypsum was identified throughout the Glacial Till and within the weathered brown-grey Oxford Clay. Forms included (Figure 5a-e) i) saccharoidal gypsum (sand-sized with a 'sugary' appearance) often on discontinuities, ii) lenses up to 10mm thick of gypsum crystals themselves 2-5mm in size, iii) isolated clusters of gypsum crystals themselves 2-20mm in size and iv) sub-spherical 'halos' up to 250mm diameter of outward-coarsening gypsum crystals surrounding a core of iron oxides (Figure 5f). Sulfides were abundant within the unweathered Oxford Clay and included i) discrete cubic pyrite crystals up to 2-3mm, ii) subrounded nodules of pyrite up to 25mm and iii) pyritised fossils. Within two weeks of excavation, 1-2mm size 'blooms' of white gypsum formed on exposed surfaces of the Oxford Clay (Figure 5g-h) whilst oxidation of pyritised fossils resulted in iron staining (Figure 5i-j).

SULFUR GEOCHEMISTRY IMPLICATIONS FOR RESERVOIR DESIGN

The DS Class was DS-4 for both the Glacial Till and Oxford Clay, i.e., the existing sulfate content is high, and there is a high potential for additional sulfate generation if sulfides oxidise. Buried concrete and metallic structures will require design to accommodate the DS Class and the relevant Aggressive Chemical Environment for Concrete Class, which itself will be a function of groundwater mobility pertaining to each structure. Field observations show that oxidation and sulfate growth commences upon exposure and therefore geochemical reactions will take place during construction of the reservoir and during its operational life. The conceptual geochemical model is summarised in Figure 6 and the processes are discussed further below.

Geochemistry considerations for the design of new earthfill embankment dams in the UK

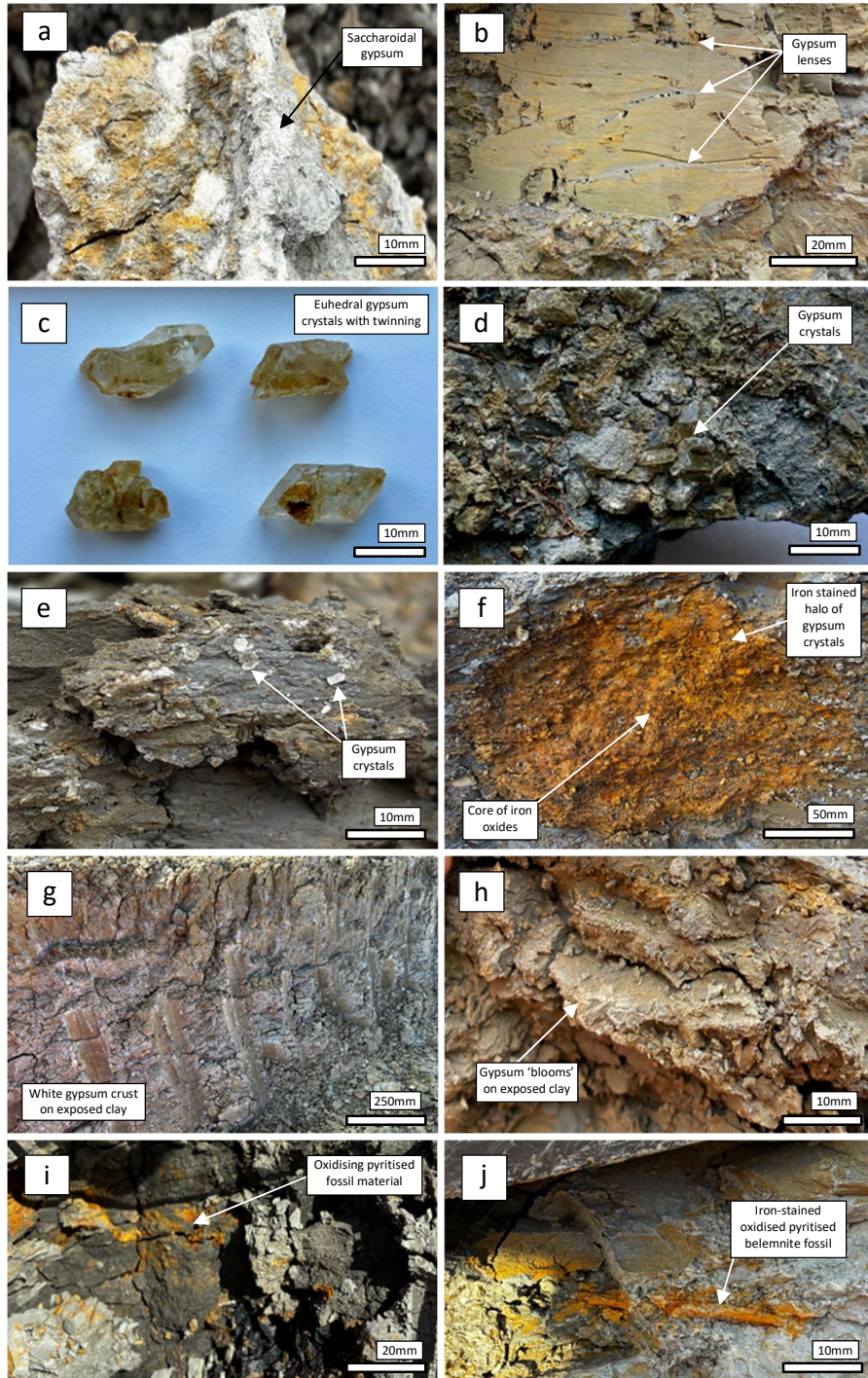


Figure 5. Sulfates and sulfides identified within the observational trial pits.

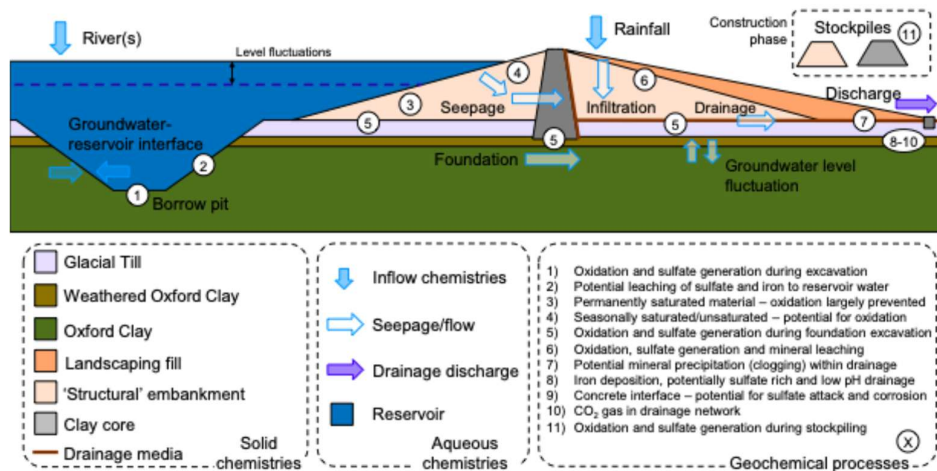


Figure 6. Conceptual geochemical ground model.

Geochemical reactions during the construction of the reservoir

The construction stage will induce the most extensive ground disturbance. Excavation to dam formation and of borrow pits will expose both sulfide-rich materials to oxygen, leading to sulfate and acidity generation. Where formation is exposed even for short periods, disturbance of the near surface material may occur due to sulfate mineral growth – leading to changes in permeability, strength and stiffness. Within clay stockpiles, oxidation will take place with sulfate growth driven by evapotranspiration, whilst rainfall infiltration will likely cause some leaching of sulfur from stockpiles. Key geochemical uncertainties to be considered ahead of design of excavations and stockpiles are i) how deep and how quickly does oxidation occur, and ii) what are the implications of the geochemical processes on the critical geotechnical and hydrogeological properties of the materials.

Material spreading prior to compaction will also induce oxidation, the extent of which will be influenced by the duration of exposure, type and amount of conditioning and the weather conditions during the works. With the bedrock clays generally dry of optimum for compaction (based upon testing reviewed to date), addition of water is expected, especially given the potentially dry conditions in eastern England during the earthworks seasons. A key geochemical uncertainty to be considered ahead of specifying the earthworks is the degree to which water conditioning influences geochemical reactions and to what extent does compaction of the clay reduce the likelihood and rate of further oxidation of sulfides to sulfates.

Geochemistry considerations for the design of new earthfill embankment dams in the UK

Geochemical reactions during the service life of the embankment

Case histories at Carsington and Roadford show that sulfides continue to oxidise decades after construction and that fill volume losses could continue. However, there are key differences between those embankments and that proposed at the Lincolnshire Reservoir. Carsington and Roadford dams were formed of indurated (i.e., rock like) material with fill permeability up to 10^{-5} m/s (Wilson and Evans, 1991). The compacted permeability of the clays at Lincolnshire Reservoir is expected to be in the range 10^{-9} to 10^{-10} m/s.

With stringent earthworks specification (including a requirement for low air voids), homogenisation of the clays and good quality control, oxidation of sulfides in high plasticity clay is likely to be self-limiting within a period of weeks post-compaction due to burial and sealing of successive fill layers (Czerewko et al, 2011). Fills that maintain saturation below the phreatic surface are unlikely to undergo further oxidation due to low oxygen diffusion and hydration of clays that maintains closure of discontinuities. A higher risk scenario is present where fills are subject to cyclic wetting and drying or dried to the extent that desiccation fissures provide preferential pathways for oxygen into the fills. In that scenario, geochemical reactions would be limited to the depth of oxygen ingress and fill volume losses would be reliant upon downstream pathways through the fill for dissolved mineral transport.

Embankment internal drainage systems reduce uncontrolled seepage and consequently reduce the likelihood of developing preferential pathways for oxygen ingress. Specification of drainage systems that might convey sulfate- and/or iron-rich seepages requires special consideration. Depending upon the chemistry of the drainage media, there is a risk of sulfate and/or iron precipitation that can lead to adverse hydraulic performance.

Many soil and rock materials processed for aggregate are calcareous (e.g., carbonates including limestone), however field and laboratory studies are recommended where carbonate contents exceed 1-5% in materials for filters (Fell, 2015). Carbonates are susceptible to dissolution by acidic seepage and typically provide a source of calcium which can react with sulfate to form gypsum. Carbonate media also raise pH, which decreases iron solubility and promotes precipitation of gel-like iron oxyhydroxides within the drainage stone. At the Lincolnshire Reservoir, the clay soils proposed for fill, and the groundwater, are already carbonate rich. Further testing and research is required to establish an appropriate limiting carbonate content for drainage aggregates, to avoid potentially precluding aggregates sources which may be more economically viable for the scheme.

FUTURE PROPOSALS FOR SULFUR GEOCHEMISTRY RISK MITIGATION

To communicate the current level of knowledge as well as the likelihood and impacts of the sulfur geochemistry hazards, a geochemistry risk register was developed. Risks and opportunities with the potential to have a major effect on construction programme or materials procurement were escalated to the project strategic risk register. Recognising the uncertainties of geochemical reaction rates and extents in construction and operation of the reservoir, further investigation was proposed as the basis for subsequent mitigation. The key proposals are described below.

Additional ground investigation and bespoke field and laboratory tests

In addition to undertaking further routine geochemical classification tests, it was proposed that geochemical profiling be undertaken at key locations. Short-interval (0.25-0.5m) testing enables improved characterisation of the vertical distribution of sulfur-species and an assessment of the practicalities of material separation in construction. Advanced petrographic assessment (XRD and scanning electron microscopy) of the clays was recommended to identify forms of sulfide present and their propensity to oxidise. Field weathering trials were recommended, comprising heaped material of each clay source subjected to periodic geochemical testing to understand the rate and degree of sulfide oxidation, sulfate generation and losses due to leaching. Laboratory testing including accelerated weathering tests such as subjecting compacted clays to acid-leaching or heating, and monitoring swell rates, was advocated to model long-term embankment chemistry changes.

Geochemical trial embankment

To complement the proposal for a main trial embankment, a separate and much smaller scale geochemistry trial embankment has been proposed, itself retaining water to induce seepage. It was recommended that a range of fill types and drainage materials be included, to assess the geochemical reactions at key material interfaces and aid the appraisal of filter aggregate sources. Additionally, a range of surface and layer finishing options (roller-sealed, vegetated, gravel-capped etc) could be considered to understand the degree of oxidation in each case. Pre- and post-trial geochemical testing and profiling, alongside specialist inspection and logging of the embankment during deconstruction was proposed, to identify the depth of oxidation and geochemical changes at key material interfaces. Periodic geochemical testing of groundwater within the embankment and foundation was also proposed to understand seasonal variations and identify potential oxidation. A minimum 12 month trial duration is proposed, with geochemical modelling and accelerated weathering tests proposed to aid forecasting of long-term fill chemistry changes, for the purposes of maintenance planning.

Geochemistry considerations for the design of new earthfill embankment dams in the UK

CONCLUSION

Sulfur-rich soils at the Lincolnshire Reservoir site will oxidise when exposed, producing sulfate, acidity, and secondary minerals such as gypsum. These reactions can affect strength, permeability, drainage, and concrete durability. Although low-permeability fills may limit long-term oxidation, uncertainties remain. Risk mitigation through further investigation, including a geochemistry trial embankment, field and laboratory tests are potential measures to aid design, material procurement and construction.

REFERENCES

- BRE (2005) *Concrete in aggressive ground*. 2nd Ed. BRE Press, Watford.
- Chalmers R W, Vaughan PR, and Coats D J (1993). Reconstructed Carsington Dam, *Proc. Inst. Civil. Eng. Water, Maritime & Energy*, **101**, 1-16.
- Cripps J C, and Edwards R L (1997) *Geotechnical problems associated with pyritic mudrocks, Ground chemistry implications for construction*. Balkema.
- Czerewko M A, Cross S A, Dumelow P G and Saadvandi A (2011). Assessment of pyritic Lower Lias mudrocks for earthworks, *Proc. Inst. Civil. Eng. Geotechnical Engineering*, **164**, 59–77.
- Fell R, MacGregor P, Stapledon D, Bell G and Foster M (2015). *Geotechnical engineering of dams*, 2nd Ed, CRC Press.
- Fasiska E, Wagenblast N and Dougherty M T (1974). Oxidation mechanisms of sulphide minerals, *Bulletin Association of Engineering Geology*, **11** 75-82.
- Hawkins A B (2014) *Engineering implications of the oxidation of pyrite, Implications of pyrite oxidation for engineering works*. Springer.
- Hopkins J K, Reid J M, McCarey J and Bray C (2010). Roadford Dam – 20 years of monitoring, In *Managing dams: Challenges in a time of change. Proceedings of the 16th Biennial Conference of the British Dam Society*. Thomas Telford, London, UK, pp 185-198
- MacDonald A and Reid J M (1991), Embankment dam behaviour: the contribution of geo-chemistry. In *Proceedings of the 6th Biennial Conference of the British Dam Society*, pp 185-191.
- Penner E, Eden W J and Gillot J E (1973). Floor heave due to biochemical weathering of shale, In *Proceedings of the 8th Conference of the International Society for Soil Mechanics and Geotechnical Engineering* (ICSMGE).

St John

- Reid J M, Czerewko M A and Cripps, J C (2005). *Sulfate specification for structural backfills*, TRL Report 447, TRL, Crowthorne, UK
- St John T W (2020), Geotechnical characterisation of sulfur species in UK Jurassic mudrocks. *Quarterly Journal of Engineering Geology & Hydrogeology* **53(4)** pp 598-608.
- Wilson A C and Evans J D (1991) The use of low grade rockfill at Roadford Dam. In *Proceedings of the 6th Biennial Conference of the British Dam Society*.



The British Dam Society

Permeability testing across scales: Consequences for dam design in low permeability clay foundations

P OTTEN, Mott MacDonald

B BOWN, Mott MacDonald

A GLOVER, Mott MacDonald

M MULHOLLAND, Mott MacDonald

M WEBB, Mott MacDonald

SYNOPSIS

Reliable characterisation of permeability in low-permeability clay foundations is fundamental to the design and construction of reservoir embankments, where permeability controls excess porewater pressure (PWP) build-up and dissipation, embankment settlement, effective stress development and thus slope stability (especially during the construction stage). However, deriving a representative design permeability is challenging due to the strong heterogeneity and anisotropy inherent in the fabric of marine clay formations. This paper presents multi-scale permeability data obtained from a current major reservoir scheme, highlighting the contrasts between laboratory scale testing (triaxial permeability, hydraulic cell, and oedometer) and field in-situ methods (variable/constant head tests and Cone Penetration Test (CPT) dissipation).

The results show that laboratory methods systematically yield lower permeability values, reflecting their small influence zones and sample disturbance effects, with hydraulic cell tests exhibiting modest anisotropy consistent with horizontal fabric in the clays. In contrast, variable head borehole tests capture a much larger soil volume. This scale dependence between methods and scale of application challenges the selection of a “characteristic” design value for embankment dams and highlights the value of large-scale testing commensurate with the scale of the proposed embankment

The paper demonstrates how uncertainty in assigning representative permeability values propagate into dam design—particularly the predicted rates of PWP dissipation during construction and first filling—and outlines how adopting a scale-aware approach, including the use of trial embankments, can improve confidence in both design parameters and performance predictions.

Permeability testing across scales: Consequences for dam design in low permeability clay foundations

INTRODUCTION

The Fens Reservoir is one of several nationally significant schemes addressing the increasing deficits in the United Kingdom's (UK) public water supply. Promoted by Anglian Water and Cambridge Water, it is progressing toward Development Consent Order (DCO) submission under the Regulators' Alliance for Progressing Infrastructure Development (RAPID) framework. Once complete, it will be one of the first reservoirs to be built in the UK for water supply since Carsington Dam in 1992. The reservoir will strengthen the UK's water supply, enhancing resilience in the east of England to drought and flooding from a future of changing climate.

The proposed reservoir lies between March (to the south) and Chatteris (to the north) in Cambridgeshire. The area is generally flat at around 0mAOD (metres above ordnance datum), rising gently towards the north western corner to a maximum elevation of 9mAOD. The proposed embankments are typically up to 12m above natural ground level, with a total perimeter length of 8.1km and reservoir water surface area of approximately 3.8km².

The geology of the Fens reflects multiple phases of change, notably during the quaternary and more recently anthropogenic drainage, creating a unique and complex geological environment. Deposition of the Oxford Clay and West Walton Formation in warm shallow seas occurred during the Jurassic Period (~170 Ma), followed by subtle tectonic movements and sediment supply variations throughout the Jurassic and Cretaceous periods. Subsequent deposition, and later erosion of Cretaceous and Eocene strata then occurred between the Jurassic and Pleistocene. Significant glaciations followed, including the Anglian and Devensian glaciations, resulting in glacial erosion, glacial landform formation as well as periglacial activity. Following the Devensian glaciation a series of marine transgressions occurred depositing the typical soft soils of the Fens. More recent anthropogenic change in the 17th century transformed the environment from marshland into fertile agricultural land.

The Bedrock comprises Jurassic Clays of the Ancholme Group (BGS, 1980); consisting of the West Walton Formation overlying the Oxford Clay Formation, with older units underlying this. These are overlain by Tidal Flat Deposits, Peat and River Terrace Deposits. The focus of this paper is on the West Walton Formation and the Oxford Clay Formation, collectively termed and referred to herein as the Jurassic Clays.

The Jurassic Clays are typically weathered to a depth of ~7.5mAOD. Where weathered, the clays are usually brown and occasionally mottled and there is gypsum present as well. Where unweathered, the material is

predominantly a stiff to very stiff dark grey slightly sandy clay, with rare fossil fragments. Fissures are often encountered, typically described as sub-horizontal to sub-vertical, very closely to medium spaced, planar and smooth. Sub-ordinate beds of predominantly extremely weak to weak grey slightly sandy siltstone are commonly present. A comparison image is included in Figure 1.



Figure 1. Colour changes identified between weathered (note the gypsum crystals within the sample) and unweathered Jurassic Clay.

The permeability of the Jurassic Clays forming the foundation is critical not only for watertightness but also strongly influences the dissipation of excess porewater pressure that is generated during construction of the earthworks and, therefore, the rate of consolidation settlement of the foundation, the strength and consequently the stability during construction. Initial desk studies and evidence from an on-site small farm reservoir suggested that the Jurassic Clays were expected to have high clay and silt content, and as a result, low intact and mass permeability. However, the fissured and laminated nature of these marine deposits, as well as the presence of the siltstone bands, give opportunity for potential increases in mass permeability. This heterogeneity and anisotropy of the Jurassic Clays means that selection of a representative design value for permeability requires careful consideration and requires data from a variety of scales.

The selection of a design permeability value should suit the scale of application. This paper discusses the impact of scale, and the selection of appropriate investigation methods to better understand mass permeability. As per standard practice within the geotechnical field, the word “permeability” is used to mean hydraulic conductivity, with units of m/s.

Permeability testing across scales: Consequences for dam design in low permeability clay foundations

METHODOLOGY

Multiple phases of ground investigation (GI) have been undertaken; the first phase between July and November 2023, and the second phase between September and December 2024. Further phases of GI are ongoing, but are not yet reported.

For the Phase 1 and Phase 2 GIs, in order to establish a suitable understanding of the permeability of the bedrock, a range of methods was carried out across both phases, as listed in Table 1. This included in-situ methods such as packer tests (BSI, 2012b), variable rising and falling head tests (BSI, 2019) and CPT dissipation tests (BSI, 2022); as well as laboratory testing including triaxial tests and oedometer.

Packer tests were generally unsuccessful as they were unable to maintain a seal with the borehole wall. Variable rising and falling head testing in the open borehole during drilling was undertaken with mixed success. One drawback of carrying out variable head testing in low permeability clay bedrock was that the groundwater level would take a significant length of time to recover, often longer than was practical to monitor, and a true rest water level was typically not established prior to the test.

However, it was found that groundwater monitoring data recorded on a data logger was a convenient way to monitor the groundwater level recovery (i.e. a rising head test) over a longer period of time once borehole construction had been completed. Standpipe piezometers were purged for two rounds of groundwater sampling, and the recovery of groundwater following this was analysed to give a value of permeability.

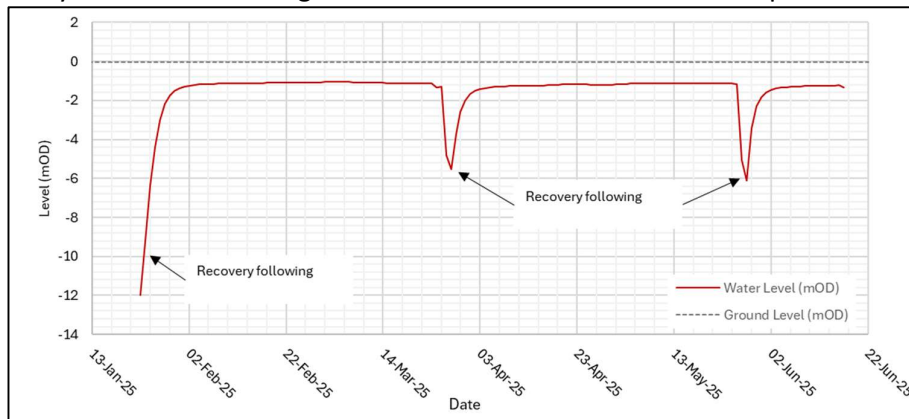


Figure 2 shows an example of this.

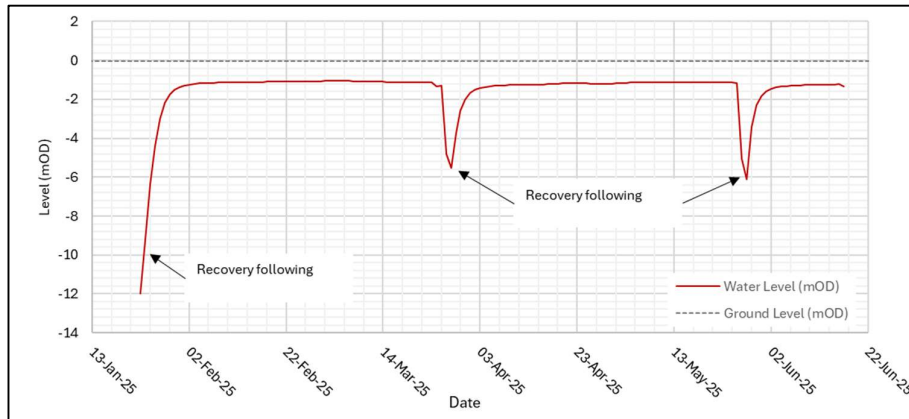


Figure 2. Groundwater monitoring of BHB1705A.

In the Phase 2 GI more focus was given to the methods found to be successful in Phase 1, with those that proved unsuccessful not attempted.

The Phase 2 GI included a mixture of rising and falling variable head tests (in standpipe piezometer installations) in response zones of varying lengths and depths. Derived values using both the Hvorslev method (BSI, 2012b) and the velocity graph method (BSI, 2021b) were compared, to conclude an appropriate value of permeability for each respective geological horizon.

Additionally, CPT dissipation tests, laboratory triaxial testing to BS EN ISO 17892-11:2019 (BSI, 2019) and hydraulic cell testing to BS1377-2:2022:20.8 (BSI, 2022) were undertaken on samples at the same depth as the installation, where possible, to allow for direct comparison of testing techniques. However, this was only practical to undertake at <10% of the locations.

RESULTS

The different methods of testing and assessment produced a wide range of permeability values. The mean and standard deviation of the results for different tests are presented in Table 1, and Figure 3 plots the same mean and standard deviation values for each test type as equivalent normal distribution curves on a logarithmic scale. The true distributions are plotted in histograms in Figure 4.

Table 1. Summary of mean and standard deviation for all tests within Bedrock Jurassic Clays

Test type	Number of tests	Number of successful tests	Geometric Mean Permeability (m/s)	Standard Deviation (applied to $\log_{10}$ of permeability)
Oedometer	39	38	1.42×10^{-11}	0.51

Permeability testing across scales: Consequences for dam design in low permeability clay foundations

(BSI, 2017)				
Triaxial permeability - vertical flow (k_v)	11	11	5.56×10^{-11}	0.61
(BSI, 2019)				
Hydraulic cell permeability - horizontal flow (k_h)	10	10	9.77×10^{-11}	0.36
(BSI, 2022)				
Variable Head test in standpipe (Bedrock)	60	57	1.81×10^{-8}	0.89
(BSI, 2012b)				
CPT dissipation	44	39	3.38×10^{-10}	0.83
(BSI, 2023)				
Borehole Packer Tests	7	0	N/A	N/A
(BSI, 2012c)				

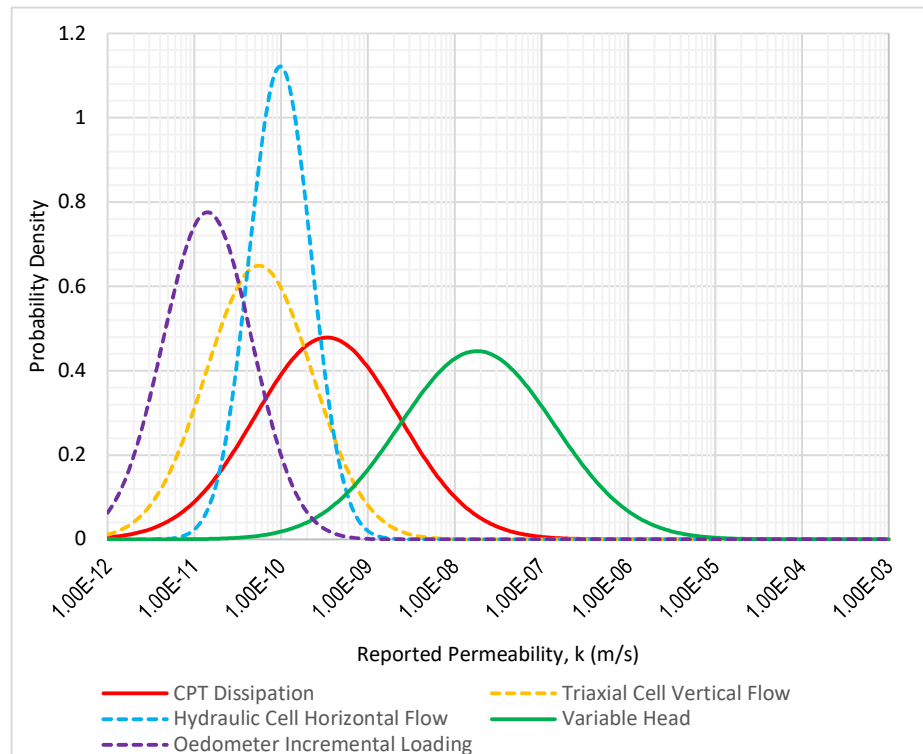


Figure 3 Normal distribution of permeability values for each testing method

The average permeability presented in Table 1 suggests that permeability in the Jurassic clays generally ranges between 1×10^{-12} and 1×10^{-7} m/s. The in-situ tests: CPT and variable head tests show the highest standard deviation

of results; however, they are also larger datasets than most of the laboratory tests. The in-situ tests also produced higher permeability values overall than the laboratory tests.

In Figure 4 it can be seen that the hydraulic cell – horizontal flow testing has a higher mean permeability compared to the triaxial cell - vertical flow testing, which indicates anisotropy within the bedrock clays. It is important to note that there were very few tests conducted of this nature, therefore this assumption cannot be definitive. The permeability value from the oedometer tests was taken from the re-loading stage and, as expected, reported a lower permeability than any other test type. This reflects the test conditions, which imposes one-dimensional vertical drainage through a laterally confined specimen at a small scale, excluding macro-scale features. Additionally, prior consolidation and re-loading reduce the voids ratio and compress the soil fabric, further restricting flow pathways.

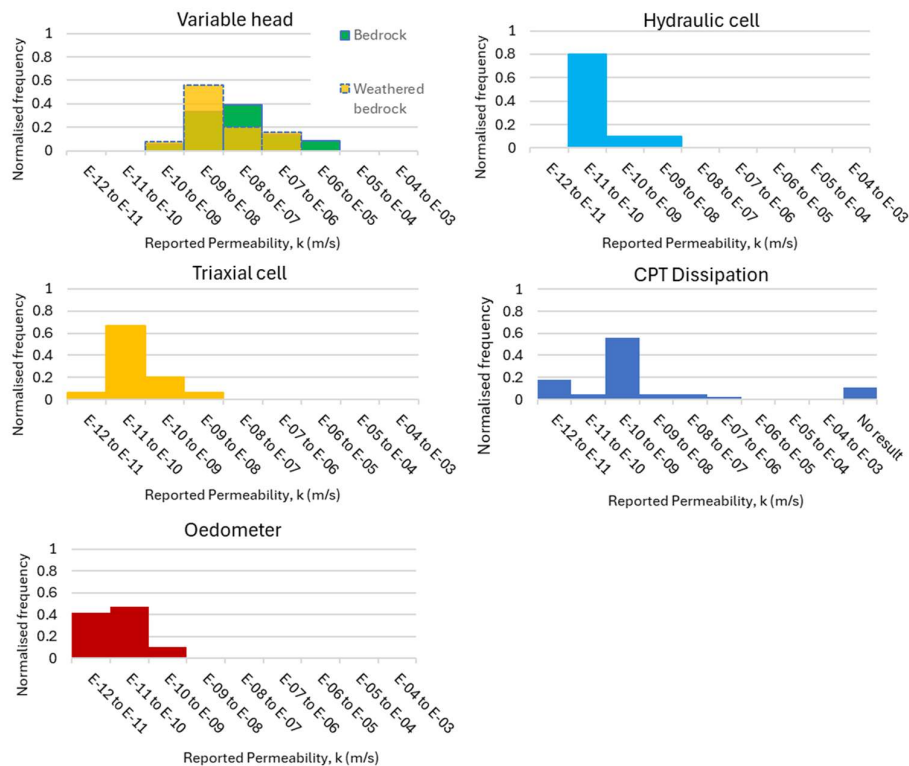


Figure 4 Histograms showing the distribution of test results by method

As outlined above, a number of locations were subject to a full suite of testing at comparable depths to enable direct comparison of permeability derivation methods, with the results consistent with the findings described above. These results are presented in Table 2. Oedometer and triaxial cell –

Permeability testing across scales: Consequences for dam design in low permeability clay foundations

vertical flow results derived the lowest permeability results, suggesting that the impact of the small sample size reduces the permeability value.

Table 2 Direct comparison of testing techniques for permeability

Location ID	Depth (mBGL)	Coefficient of Permeability (m/s)			
		Triaxial Cell	Hydraulic Cell	Standpipe Permeability Test	Co-located CPT
BHA2102A	8.0-11.0	5.2×10^{-11}	8.6×10^{-11}	3.38×10^{-7}	1.38×10^{-10}
BHB1705A	12.0-15.0	4.8×10^{-11}	1.2×10^{-10}	$2.54 \times 10^{-8*}$	N/A
BHB1708	7.0-9.0	1.6×10^{-11}	1.0×10^{-9}	$5.53 \times 10^{-6*}$	N/A

*Average taken over 3 separate monitoring rounds.

There was no clear correlation between depth or level of sample or test and reported permeability for any of the applied methods. This means that despite the clear distinction between the weathered and unweathered Jurassic Clay in hand specimens, it was not possible to distinguish these materials based on the results of permeability testing.

IMPLICATIONS FOR DAM DESIGN

The permeability of foundation clays plays a significant role in the behaviour of the embankment during construction and operation. Therefore, reliable characterisation of permeability is fundamental to the design of reservoir embankments, where permeability controls excess porewater pressure build-up and dissipation, embankment settlement, effective stress development and thus slope stability. Given that permeability influences these mechanisms differently, and in some cases inversely (for example, higher permeability may be conservative for leakage but not for excess pore water pressure during construction), a range of potential permeabilities should be applied to determine the least favourable combination in any particular design case.

As demonstrated within this paper the reported permeability for the Jurassic Clays is significantly dependant on the scale and type of testing, a reflection of their inherent heterogeneity and anisotropy, stress history and depositional environment.

Figure 6 presents settlement profiles over time for a range of typical permeabilities reported during the Ground Investigations, illustrating how permeability influences settlement behaviour in different ways for the proposed Fens Reservoir. For lower permeability values, a significant proportion of consolidation settlement occurs post-construction; therefore, this settlement must be accounted for, notably ensuring freeboard is

maintained. Conversely, for higher permeability values, a greater proportion of consolidation settlement occurs during construction, reducing the impact on post construction freeboard allowances. This further reinforces that no single representative permeability value can adequately represent all design considerations.

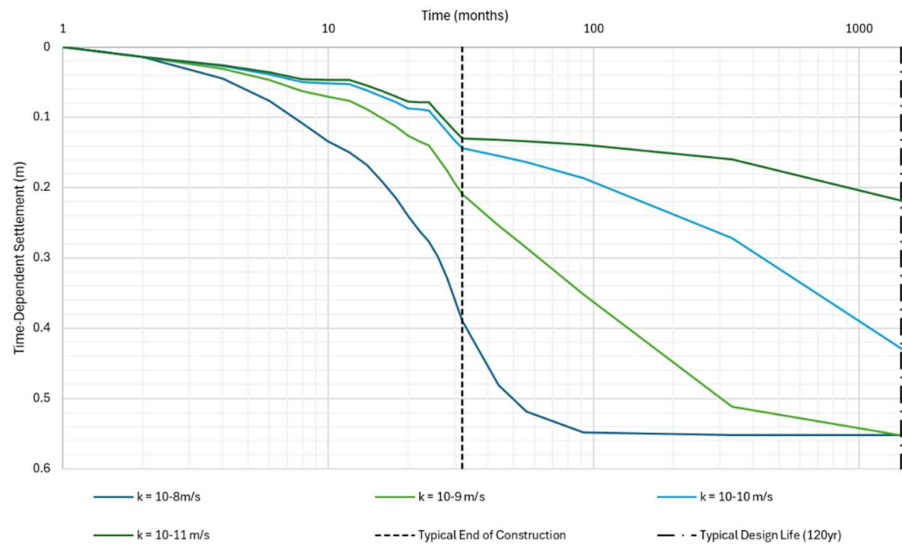


Figure 6 Consolidation settlement with time for various permeabilities against typical milestones.

There remains inherent uncertainty in the characterisation of permeability; however, its selection exerts a critical and often highly variable influence on key design mechanisms, including leakage, settlement behaviour, and porewater pressure dissipation. These responses may act in opposing ways, meaning that no single permeability value can be regarded as universally conservative. This reinforces the need for a more robust, evidence-led approach. To meaningfully reduce uncertainty and support mechanism informed design, large-scale trials, specifically a fully instrumented trial embankment, are important. Such trials provide the only reliable means of defining representative mass permeability, which ultimately governs both foundation response and overall embankment performance.

DISCUSSION

Multiple techniques to measure permeability were undertaken in the two phases of GI, as described above, with the methods selected chosen based on cost efficiency as part of conventional GI. However, neither a pumping test nor a trial embankment were carried out at this stage. These two methods are highly effective for determining large scale permeability, however, there are limitations to pumping tests within low permeability

Permeability testing across scales: Consequences for dam design in low permeability clay foundations

materials and they were not deemed suitable for the site. It will be necessary for a trial embankment and it is recommended that this takes place prior to the start of construction, which will help to further constrain bedrock permeability.

As pore-scale permeability is expected to be very low, the dominant influences on the mass permeability of the Jurassic Clay are expected to be fissures and layers of siltstone.

Anisotropy of the clay has been weakly indicated by the triaxial and hydraulic cell results. However, the sample size is considered too small to conclusively show anisotropy, and would require further investigation.

It was also expected that the weathered Jurassic Clay bedrock may exhibit a different permeability due to the alteration of the rock fabric (including, for example, swelling of clay minerals and closing of fissures, or flow paths for groundwater becoming better established). However, from the results there was no clear distinction between the permeability results for the weathered and unweathered Jurassic Clay.

It should be noted that as well as scale effects producing the difference between in-situ and laboratory testing, the laboratory tests may also have been influenced by disturbance of the samples, however carefully taken, preserved and transported. This may have affected the permeability of the sample.

The wide range of potential values for permeability over several orders of magnitude is implicit with the natural variation within the ground, even for materials which in descriptions and hand samples, look to be relatively homogeneous. The presence and joined up or disconnected nature of fissures and siltstone beds makes permeability at the tested scales highly variable.

The natural variation of the permeability of the ground is expected to average out to some extent over the area of the embankment foundation. While there is potential for different parts of the site to behave differently, there is no clear spatial trend in permeability, either horizontally across the site, or with depth. It is therefore considered appropriate to use narrower ranges of permeability for design than the full range of test values.

CONCLUSION

The GI completed to date at the Fens Reservoir illustrates how uncertainty in permeability can be progressively reduced through a staged, multi-scale approach. At the investigation stage, the most reliable field-based methods for assessing bulk behaviour were found to be variable head tests in

standpipes and dissipation testing from CPTs, while triaxial and hydraulic cell laboratory tests provided valuable insight at the smaller, pore-scale level. These investigations demonstrate that permeability in the Jurassic Clays is scale-dependent. At the pore scale, the material exhibits low permeability with indications of slight anisotropy. However, as the scale increases, permeability is increasingly governed by discontinuities and natural variability, resulting in heterogeneity spanning several orders of magnitude. No consistent spatial trends or clear distinction between weathered and unweathered bedrock were identified, reinforcing the inherent variability of the material.

Permeability of foundation clays is therefore a critical yet uncertain parameter that evolves in definition as a project progresses. It controls key aspects of embankment behaviour such as pore pressure dissipation, effective stress development, settlement and stability and these responses can act in opposing ways dependant on the assumed value. The observed variability between laboratory, in situ, and larger-scale tests highlights that uncertainty cannot be resolved through a single “conservative” parameter. Instead, different permeability ranges must be considered for different mechanisms: lower values leading to slower consolidation and higher construction pore pressures, and higher values promoting earlier settlement but reducing long-term impacts.

Accordingly, ground investigation should be viewed as an iterative process in which uncertainty is progressively reduced by moving from laboratory testing and conventional in situ measurements, ultimately to large-scale verification. Ground investigation should be proportionate to the engineering complexity of the project so for a smaller project, a single phase of GI followed by observation during construction may be appropriate. For Fens Reservoir, the investigations completed and discussed within this paper highlight the importance of large-scale verification such as trial embankments to establish representative mass permeability. Such staged validation enables refinement of design parameters and provides the level of confidence required to manage risks associated with leakage, settlement, and stability, rather than relying solely on early-stage test results.

ACKNOWLEDGMENT

The authors would like to thank Anglian Water and Cambridge Water for permission to publish this paper, and to colleagues in Mott MacDonald for assistance and support.

Permeability testing across scales: Consequences for dam design in low permeability clay foundations

REFERENCES

- BGS (1980) *Ely. England and Wales Sheet 173. Solid and Drift Geology. 1:50,000 Provisional Series*. British Geological Survey, Keyworth, Nottingham
- BSI (2012a) *BS EN ISO 22282-1:2012 Geotechnical investigation and testing - Geohydraulic testing Part 1: General Rules*. BSI Standards Limited.
- BSI (2012b) *BS EN ISO 22282-2:2012 Geotechnical investigation and testing - Geohydraulic testing Part 2: Water permeability tests in a borehole using open systems*. BSI Standards Limited.
- BSI (2012c) *BS EN ISO 22282-3:2012 Geotechnical investigation and testing. Geohydraulic testing. Water pressure tests in rock*. BSI Standards Limited.
- BSI (2017) *BS EN ISO 17892-5:2017 Geotechnical investigation and testing. Laboratory testing of soil. Incremental loading oedometer test*. BSI Standards Limited.
- BSI (2019) *BS EN ISO 17892-11:2019 Geotechnical Investigation and testing - laboratory testing of soil. Part 11: Permeability tests*. BSI Standards Limited.
- BSI (2022) *BS 1377-2 - Methods of test for soils for civil engineering purposes, Part 2: Classification tests and determination of geotechnical properties*. BSI Standards Limited.
- BSI (2023) *BS EN ISO 22476-1:2023 Geotechnical investigation and testing. Field testing. Electrical cone and piezocone penetration tests*. BSI Standards Limited.



Applying the Reservoirs Act 1975 to specks in the Atlantic Ocean

J HOWARD, JBA Consulting

J BENN, JBA Consulting

SYNOPSIS

UK Reservoir legislation provides a mature, risk-based framework for the regulation and management of large raised reservoirs, supported by well-established technical guidance, professional roles, and inspection regimes. This paper explores the application of the principles of UK legislation to British Overseas Territories, using recent investigations of the reservoir portfolio on St Helena and for reservoirs on South Georgia.

Although the Reservoirs Act 1975 formally applies on St Helena, it does so without the subsequent amendments, secondary legislation, and supporting institutions that underpin current UK practice. In South Georgia the Act does not formally apply but is used as the basis for 'good practice'.

The paper examines how core elements of the Act - risk-based classification, qualified engineer oversight, prescribed inspections, and proportional remedial action - can be applied pragmatically in small, remote, resource-constrained jurisdictions. Detailed consideration is given to the Harpers 2 Reservoir seepage investigation, flood hydrology review, and reservoir risk assessment, which were undertaken broadly in line with contemporary UK guidance despite significant limitations in data availability, local construction capability, and regulatory infrastructure.

The wider inspection of the St Helena and South Georgia reservoir portfolios highlights common challenges, including ageing assets, informal operational practices, and the absence of systematic records.

The paper concludes that the Reservoirs Act provides a robust conceptual framework that can be successfully transferred internationally, but that effective implementation requires flexibility, proportionality, and adaptation to local institutional capacity, climate, and socio-economic context.

INTRODUCTION

British Overseas Territories (BOTs) present markedly different operating environments to the UK: small dam portfolios, remote locations, limited specialist capacity, and high logistical costs. Nevertheless, many territories

Applying the Reservoirs Act 1975 to specks in the Atlantic Ocean

rely on reservoirs as critical infrastructure for water supply, energy generation, or heritage protection. This paper explores how the principles embedded in UK reservoir legislation can be applied pragmatically in such contexts, focusing on recent work on St Helena and South Georgia.

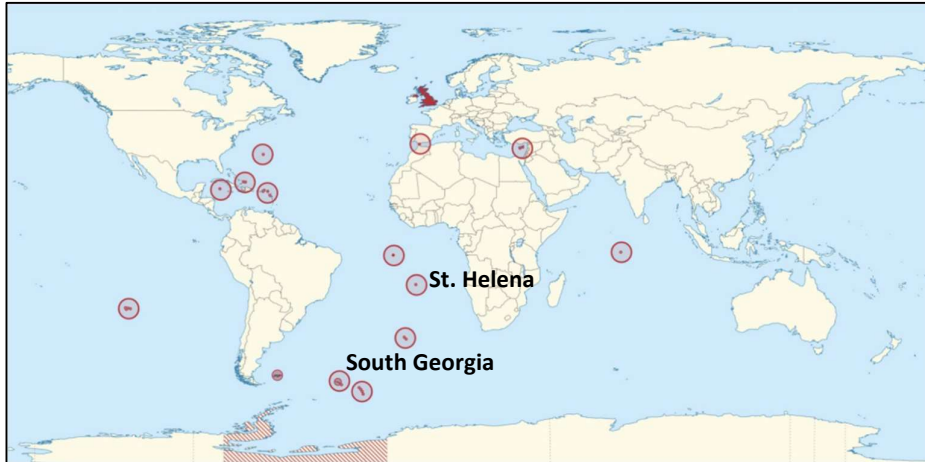


Figure 1. Location of British Overseas Territories

St. Helena

St. Helena is a remote volcanic island in the South Atlantic Ocean with a land area of 122 km² (approximately the size of Bristol). and a resident population of approximately 4,400. It lies approximately 1,100 km from the nearest land at Ascension Island, around 1,950 km west of the African mainland, and roughly 4,000 km east of South America (Figure 1), making external access and emergency support inherently constrained. It is a UK Overseas Territory administered by the St. Helena Government under a Governor appointed by the Crown, with a locally elected Legislative Council responsible for domestic policy and infrastructure management.

Land use is dominated by steep terrain and environmentally sensitive areas with agriculture undertaken on a small scale, primarily for local consumption and livestock grazing. The main population centre is located around Jamestown with numerous small communities distributed across the island.

Until 2016 the island had no airport and access for people and supplies was via a weekly supply ship to and from Cape Town. Even today there is only one weekly flight from Johannesburg and a local connection to Ascension Island.

Reservoirs form critical national infrastructure for potable water supply due to the limited natural storage, highly variable rainfall (annual rainfall is between 110 mm and 1,000 mm depending on location), and the absence of any large rivers.

South Georgia

South Georgia is a remote sub-Antarctic island in the South Atlantic Ocean, located approximately 1,400 km east-south-east of the Falkland Islands and over 2,000 km from the nearest continental landmass. The island is a UK Overseas Territory, administered as part of South Georgia and the South Sandwich Islands (SGSSI), with governance exercised by a Commissioner based in the Falkland Islands. It has a land area of 3,903 km² (approximately twice the size of Greater London). There is no permanent indigenous population. The only settlements consist of small, intermittently occupied British Antarctic Survey (BAS) research stations, principally King Edward Point near the abandoned whaling station of Grytviken, with additional seasonal facilities at locations such as Bird Island. South Georgia has a cold, maritime sub-Antarctic climate, characterised by frequent cloud, strong winds and precipitation throughout the year, with annual rainfall (and snowfall equivalent) typically of the order of 1,000–1,500 mm, and mean temperatures generally ranging from around –2 °C in winter to 8 °C in summer. Glaciers and permanent snow cover dominate much of the island's interior.

The island has no airstrips or scheduled boat connection. Supplies and personnel arrive and leave by either chartered ship from Port Stanley in the Falkland Islands or by BAS or Royal Navy ships. Travel from the UK involves using the Falkland Islands 'Air Bridge' via Ascension Island and then a 2-3 day journey by chartered ship.

REGULATORY AND INSTITUTIONAL CONTEXT

The Reservoirs Act 1975 (RA1975) underpins dam safety practice in England and Wales, establishing a proportionate, risk-based regulatory framework for large raised reservoirs. Similar legislation exists in Scotland and Northern Ireland. The Acts effectiveness derives not only from primary legislation, but also from supporting regulations, principally those introduced under the Flood and Water Management Act 2010, including the Reservoirs Act 1975 (Exemptions, Appeals and Inspections) (England) Regulations 2013 and related statutory instruments and from a substantial body of technical guidance. This includes the ICE Guide to the Reservoirs Act 1975 and Floods and Reservoir Safety series, together with Environment Agency guidance covering inspection, risk assessment, flood planning and operational management of reservoirs, such as the Guide to Risk Assessment for Reservoir Safety Management and the Guide to Drawdown Capacity for Reservoir Safety and Emergency Planning. A system of qualified reservoir engineers operating in defined statutory roles supports implementation.

Applying the Reservoirs Act 1975 to specks in the Atlantic Ocean

Reservoir Regulation in British Overseas Territories

On St Helena, Connect Saint Helena Ltd (CSH) operates and maintains the island's water infrastructure, including the full reservoir portfolio, and in practice fulfils the role of undertaker for reservoir safety management. Reservoir safety is governed primarily through local water legislation. While the RA1975 applies in principle, local legislation has not been updated to reflect subsequent UK amendments, nor is it supported by the full suite of secondary regulations, statutory forms, or enforcement mechanisms present in the UK. In the absence of a fully developed statutory framework, implementation relies on the adoption of UK good practice by CSH, supported by periodic input from UK-qualified reservoir engineers.

In South Georgia and the South Sandwich Islands, there is no reservoir-specific legislation. Dam safety is instead managed through general government duties, with reliance on external technical assurance and adherence to recognised international and UK standards.

Implications for Practice

In both territories, reservoir safety management relies on:

- Adoption of UK technical guidance as non-statutory good practice
- Engagement of UK-qualified reservoir engineers on a periodic basis
- Proportionate application of inspection, monitoring and remedial measures aligned to actual risk and available resources

In practice, the application of RA1975 principles in British Overseas Territories departs from formal UK implementation, requiring a pragmatic, risk-led approach aligned to local constraints. The following case study from St Helena demonstrates how these principles are translated into practice at both portfolio and asset scale, highlighting the challenges of maintaining dam safety within a small, remote and resource-constrained system.

CASE STUDY: ST HELENA – HARPERS 2 RESERVOIR

Background

The St Helena reservoir system provides a representative example of how RA1975 principles are applied in a small, remote and resource-constrained context. The portfolio comprises a combination of non-impounding lined reservoirs and a single large impounding earth embankment dam at Harpers 2, which represents the highest consequence asset within the system. (Figure 2 and Table 1)

The island's steep terrain and limited opportunities for large-scale storage necessitate a distributed water supply system comprising multiple small reservoirs linked by pipelines and transfer infrastructure. This configuration creates a high degree of interdependence between assets, such that the performance or failure of a single reservoir can have system-wide implications.

Within this setting, reservoir safety management relies on the pragmatic adaptation of UK practice, including constrained access to data, limited construction capability, and the absence of a comprehensive statutory framework. Implementation is therefore necessarily proportionate, with emphasis placed on risk-informed decision making, prioritisation of critical assets, and periodic input from UK-qualified reservoir engineers rather than continuous regulatory oversight.

In recent years, St Helena has experienced periodic drought conditions, with water restrictions imposed in March 2013, November 2016 and June 2019, each of which remained in place for several months. Within this context, works were undertaken in 2016 to raise the spillway crest of Harpers 2 Dam to increase storage capacity and improve water security on the island. However, limited information is available regarding the scope, design basis and supervision of these modifications, and no detailed records have been identified. This absence of formal documentation represents a significant gap in understanding the behaviour of the dam and contrasts with the structured design, supervision and certification processes typically associated with UK reservoir practice.

Importantly, these works were driven by the operational need to increase water storage in response to drought risk, reflecting the pressures faced by small, resource-constrained systems. This highlights how the imperative to improve water security can, in the absence of formalised regulatory oversight and structured engineering assurance, lead to incremental changes that increase dam safety risk over time.

In September 2024, seepage was identified on the downstream face of the embankment, prompting precautionary drawdown and initiation of a detailed safety investigation.

Applying the Reservoirs Act 1975 to specks in the Atlantic Ocean

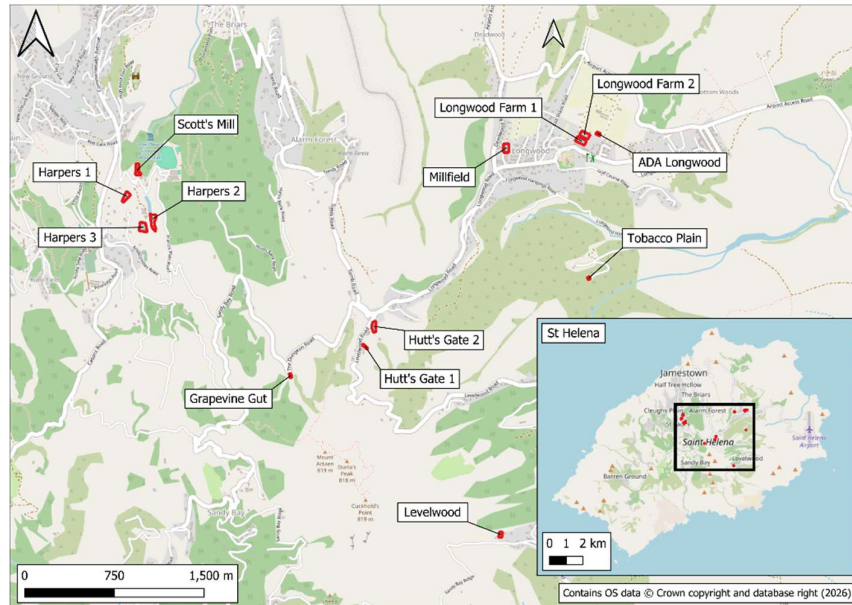


Figure 2. Location of reservoirs on St. Helena

Table 1. Details of reservoirs on St. Helena

Name	Dam Height/ Capacity	Description
Grapevine Gut	4.5 m/ 1,301 m ³	Impounding earth dam with HDPE liner. Construction date unknown.
Harpers 1	8.0 m/ 11 ,512 m ³	Non-impounding earth dam with HDPE liner. Constructed 1980's.
Harpers 2	9.0 m/ 30,000 m ³	Impounding earth embankment dam constructed 1994.
Harpers 3	9.0m/ 13,633 m ³	Non-impounding earth dam with HDPE liner constructed 1995 and subsequently significantly enlarged.
Hutts Gate 1	5.0 m/ 2,090 m ³	Non-impounding earth dam with HDPE liner. Constructed 1983.
Hutts Gate 2	9 m/ 9,730 m ³	Non-impounding earth dam with HDPE liner. Constructed 2017-18.
Levelwood	2.0 m/ 2,711 m ³	Non-impounding earth dam with HDPE liner. Construction date unknown.
Longwood 1	4.5 m/ 9,661 m ³	Non-impounding earth dam with HDPE liner. Construction date unknown.

Name	Dam Height/ Capacity	Description
Longwood 2	4 m/ 6260 m3	Non-impounding earth dam with HDPE liner. Construction date unknown.
Longwood 3	2.0m/ 339 m3	Non-impounding earth dam with HDPE liner. Construction date unknown.
Millfield	5.0 m/ 8867 m3	Non-impounding earth dam with HDPE liner. Construction date unknown.
Scotts Mill	4.5 m/ 6887 m3	Non-impounding earth dam with HDPE liner constructed 1981.
Tobacco Plain	2.0 m/ 542 m3	Non-impounding earth dam with HDPE liner. Construction date unknown.

Technical Investigations

Investigations undertaken broadly followed UK practice under RA1975:

- Detailed site inspection by a UK Panel Engineer
- Review of historical design, construction and modification records
- Assessment of seepage mechanisms, including inspection of drainage and foundation conditions
- Review of flood hydrology and freeboard relative to contemporary guidance
- Tier 2 Reservoir Risk Assessment (RARS) undertaken in accordance with Environment Agency guidance

Seepage concerns were first identified following the spillway crest raising works. The timing of these observations, together with the increased hydraulic loading associated with higher reservoir levels, raised concerns that the modifications may have influenced seepage behaviour within the structure. Subsequent investigations indicated that seepage was more likely associated with blocked drainage and permeable bedrock beneath the spillway, rather than directly attributable to increased hydraulic head following the 2016 modifications.

Risk Assessment and Optioneering

Application of RARS demonstrated that, under “rainy day” conditions, the reservoir risk lay on the boundary between ALARP and Unacceptable, driven primarily by flood overtopping and internal erosion failure modes.

Applying the Reservoirs Act 1975 to specks in the Atlantic Ocean

A multi-criteria analysis was used to assess remedial options, considering safety benefit, constructability, environmental impact, cost, and logistical feasibility. In line with RA1975 principles of proportionality, a staged approach was recommended:

- Maintenance and enhancement of drainage, installation of a filter drain, and improved monitoring to allow safe return to service
- Contingent implementation of more intrusive seepage barriers if monitoring indicates ongoing risk

This approach enabled rapid risk reduction while maintaining flexibility and affordability. While CSH has established a comprehensive monitoring regime, recommendations were made to enhance data capture, record keeping and trend analysis to ensure that future inspections are supported by a robust and continuous evidence base for decision-making.

CASE STUDY: SOUTH GEORGIA RESERVOIRS

Asset Overview

South Georgia hosts small reservoirs at Bore Valley and Gull Lake, originally constructed c.1912–1914 and commissioned in 1914 to support whaling operations and later adapted for water supply and hydroelectric use for the BAS Research Station at King Edwards Point. These assets are located close to the eastern coast of the island close to the former whaling station of Grytviken (Figure 3 and Table 2).



Figure 3. Location of reservoirs at Grytviken on South Georgia

Gull Lake

The original dam wall was constructed in 1913 to provide hydroelectricity to the Grytviken whaling station. The dam wall was raised in 1928.

In 2007, Morrison Construction was commissioned to construct a new facility, encasing the original dam wall within the new structure as well as replacing the supply pipeline and hydroelectric plant that serve the BAS research station and Grytviken facilities. Since its construction, some crack repairs along the dam wall have taken place. As-built records are kept at the research station.

Bore Valley

The impoundment consists of a 3m high and 10m long RC wall founded on rock. Water is abstracted via a penstock located in a chamber integral to the dam. Water is then conveyed via a 550 m long 200mm OD cast iron pipe to Grytviken where it can be used to supply the nearby BAS station. It formally also supplied a small turbine. The depth of cover of the pipe varies from 0m to c.1m. It is believed there is no selected backfill or bedding layer to the pipe trench.

Table 2. Details of reservoirs on South Georgia

Name	Dam Height/ Capacity	Description
Bore Valley	4.5 m/ c.2,000 m ³	Constructed in 1920s for water supply and hydro-power but currently not used for either purpose. Mass concrete dam with integral spillway.
Gull Lake	3.8 m/ 440,000 m ³	Originally constructed in 1913-14 and substantially rebuilt in 2008. Water from Gull Lake supplies a c.260 kW Gilkes turbine and provides water supply to Grytviken and King Edward Point. Reinforce concrete dam with integral spillway

Condition Assessments

Visual condition surveys in 2020 and 2025, broadly equivalent to Section 10 inspections under RA1975, identified:

- Ageing concrete and masonry structures with patch repairs and weathering
- Limited or absent as-built records

Applying the Reservoirs Act 1975 to specks in the Atlantic Ocean

- Deterioration of ancillary assets such as penstocks, chambers and ladders
- Access and safety constraints for inspection and maintenance activities

Application of RA1975 Principles

Although no statutory framework exists, the following RA1975 principles are applied:

- Independent inspection by suitably qualified engineers
- Clear identification of credible failure modes and consequences
- Prioritisation of remedial works addressing life-safety and asset integrity
- Recommendations for proportionate inspection intervals and record keeping

The assessments demonstrated that, even in the absence of legislation, the conceptual framework of the Act provides a consistent and defensible basis for managing dam safety risk. These assets demonstrate similar themes to St Helena, including aged assets, incremental modification, and dependence on limited records and local capability.

DISCUSSION

Transferability of the Reservoirs Act Framework

The case studies illustrate that the strength of RA1975 lies in its structure rather than its prescriptiveness. Key transferable elements include:

- Risk-based classification focused on consequences rather than size alone
- Use of qualified, independent engineers
- Emphasis on monitoring, records and learning over time
- Proportionate, staged intervention aligned to risk

Across both territories, recurring challenges include limited historical records, ageing infrastructure, constrained access for inspection and maintenance, and reliance on informal operational practices.

Adaptation to Local Context

Effective reservoir safety in the BOTs requires adaptation from established UK practice, reflecting both physical and institutional constraints. On St Helena in particular, the distributed nature of the water supply system

comprising multiple small reservoirs linked by pipelines across steep terrain creates a high degree of interdependence between assets.

In this context, the consequences of failure are not limited to individual structures, but extend to system-wide impacts on water supply resilience and downstream communities. This fundamentally reinforces the need for a proportional approach to dam safety management, in which risk reduction measures are prioritised based on consequence, feasibility and overall system benefit rather than strict compliance with prescriptive standards.

Practical implementation therefore emphasises targeted intervention at critical assets, staged risk reduction, and the use of observational and performance-based approaches where data are limited. This contrasts with the more uniformly codified application of the Reservoirs Act framework in the UK, but is consistent with its underlying risk-based principles.

First, there is recognition of the disproportionate consequences of failure in isolated communities. On St Helena, failure of key reservoirs would have immediate and significant implications for potable water supply, with limited system redundancy and constrained ability to provide emergency supply. For Harpers 2, located above Jamestown, an uncontrolled release would pose direct risk to the island's capital, including potential loss of life, damage to critical infrastructure, and disruption to essential services such as government administration and healthcare. These impacts would be proportionally greater than for a similar event in the UK, reflecting the island's remote location and the limited redundancy of critical infrastructure. In South Georgia, dam breach and uncontrolled release of water would impact the heritage site of the Grytviken whaling station and the cemetery which includes the last resting place of Sir Ernest Shackleton. Loss of power from the turbines would also affect the BAS station and its complement of approximately 22 researchers and their scientific activities.

Second, there is acceptance of limited data and reliance on observational methods, alongside flexibility in inspection frequency due to access constraints. In South Georgia, ground-based inspection is only possible during the Antarctic summer, further reinforcing the need for adaptive monitoring approaches.

Third, optioneering must prioritise constructability using locally available skills and materials, or those that can realistically be mobilised to site.

Fourth, the fragile and unique natural environments in these territories require exceptionally high standards of biosecurity. For example, importing construction materials such as aggregates requires careful management to minimise the risk of introducing non-native species.

Applying the Reservoirs Act 1975 to specks in the Atlantic Ocean

In such settings, proportionality is not simply a matter of practicality, but a necessary condition for achieving sustainable and defensible dam safety outcomes.

CONCLUSIONS

The Reservoirs Act 1975 provides a robust conceptual framework that can be successfully applied outside the UK, including in small, remote British Overseas Territories. The experiences from St Helena and South Georgia demonstrate that, even where formal legislation is absent or incomplete, the Act's principles support effective, proportionate reservoir safety management.

However, successful implementation depends on flexibility, professional judgement and adaptation to local institutional capacity and environmental context. For BOTs with critical water infrastructure, structured adoption of RA1975 principles represents a practical and defensible pathway to maintaining dam safety.

REFERENCES

- Balmforth D (2021). *Independent Reservoir Safety Review Report*, UK Government.
- Environment Agency (2013). *Guide to Risk Assessment for Reservoir Safety (RARS)*. Environment Agency, Bristol, UK
- HM Government (1975). *Reservoirs Act 1975*. The Stationery Office, London, UK
- HM Government (2010). *Flood and Water Management Act 2010*. The Stationery Office, London, UK
- HM Government (2013). *Reservoirs Act 1975 (Exemptions, Appeals and Inspections) (England) Regulations 2013*. The Stationery Office, London, UK
- ICE (2019). *Guide to the Reservoirs Act 1975*. Institution of Civil Engineers, London, UK
- ICOLD (2009). *Bulletin 138: General Approach to Dam Surveillance*. International Commission on Large Dams, Paris, France.
- ICOLD (2014). *Bulletin 154: Dam Safety Management – Operational Phase of the Dam Life Cycle*. International Commission on Large Dams, Paris, France
- JBA Consulting (2025). *Report of an inspection of Harpers 2 Reservoir*
- JBA Consulting (2025). *Report of an inspection of the CSH Reservoir Portfolio*



The British Dam Society

Strengthening Dam Emergency Preparedness: Lessons learnt from British Columbia, Canada

K MILLER, WSP

SYNOPSIS

As the UK prepares for changes under the Reservoir Safety Reform Programme, strengthening emergency preparedness is increasingly important.

Drawing on practical experience from dam emergency management in British Columbia, Canada, this paper provides an overview of dam emergency plans designed to meet the BC Dam Safety Regulation and explores how realistic exercises, meaningful stakeholder engagement, and lessons learned from minor emergency events can enhance readiness and build confidence in emergency procedures. In addition, it explores how public communication of flood maps can be integrated into emergency planning.

The dam emergency plans adopted by many dam owners in BC include a structure that provides clear actions: Event detection, determine emergency response level, notification and communication, expected actions and termination of the emergency and follow up. The response levels are broken into three levels: Level 1 where there is an unusual event that is slowly developing, Level 2 where there is potential for failure that is rapidly developing, and Level 3 is an urgent situation where failure is imminent or in progress.

Practical exercises play a critical role in strengthening the plans, including confirming that roles and responsibilities are well understood, communication systems function as intended, and essential information can be accessed quickly during an emergency. The same can be said about minor incidents. Each exercise or small-scale event provides an opportunity for structured debriefing, which helps bridge the gap between the written plan and the real-world execution of emergency actions.

INTRODUCTION

British Columbia is one of four provinces in Canada with a formalised dam safety program. The dam safety program in British Columbia is risk-based and aligns with the principle that dam owners strive to reduce risks to a level that is as low as reasonably practicable (ALARP) (BC, 2024). The regulation provides a framework for owners to establish a dam safety management

Strengthening Dam Emergency Preparedness: Lessons learnt from British Columbia, Canada

system including an operation, maintenance and surveillance manual, dam emergency response, dam safety reviews, and requirements to submit engineering designs for dam alterations.

Emergency response is a key component of dam safety management in British Columbia and is formalized through dam emergency plans prepared to meet the requirements of the BC Dam Safety Regulation. A dam emergency plan is intended to facilitate a safe, timely, effective and coordinated response during a dam emergency event.

Dam Safety Program In British Columbia

BC's dam safety program began in 1967 in response to the rapid development of large hydroelectric dams in BC and the first BC Dam Safety Regulation was brought into force in 2000 based on recommendations of an independent program review of the 1995 Cannon Creek Dam failure. The regulation placed legal responsibility for the safety of dams on dam owners, requiring owners to operate, maintain, and inspect dams to minimize risk to the public, the environment, and the economy (BC, 2016). On February 29, 2016, the Water Sustainability Act and Dam Safety Regulation came into force replacing the former Water Act and BC Dam Safety Regulation.

The Dam Safety Regulation governs dam owners' responsibilities in safety management. The objective of the Regulation is to mitigate loss of life and damage to property and the environment from a dam breach by requiring dam owners to inspect their own dams, undertake proper maintenance on them, and ensure that these dams meet engineering standards to minimise associated public safety, economic, social, and environmental risks (BC, 2024).

A regulated dam is a dam that is greater than 7.5 m in height and is able to impound 10,000m³ of water or more (BC DS Regulation). A dam that is below this requirement may be regulated if there is a perceived risk to public safety, the environment, or land or other property. There are currently 1,914 regulated dams in BC, the purpose of the dams including hydropower (hydropower sources 87% of the electricity in BC), water supply, wildlife conservation, industrial use, recreation, and flood control.

Regulated dams are characterised by their failure consequence classification which is based on the potential loss of life, deterioration of environmental and cultural values, and losses to infrastructure and economy if the dam were to fail. Regulated dams are split into the following categories; low, significant, high, very high and extreme.

Overview of the BC Dam Emergency Plan Guidelines

The BC Ministry of Water, Land and Resource Stewardship provides instructions, guides and a template for preparing a Dam Emergency Plan in British Columbia. The template is adapted from the Canadian Dam Association guidelines (CDA, 2013) which provide guidance on emergency preparedness for dam owners, local emergency authorities, and communities. The Dam Emergency Plan template contains basic dam information, details of access to the dam, roles and responsibilities, emergency contacts, evacuation area maps, list of persons in the immediate vicinity, and a 5 Step process which includes the following:

- Step 1: Event detection,
- Step 2: Determination of the emergency response level
- Step 3: Notification and communication,
- Step 4: Expected actions
- Step 5: Termination of the emergency and follow-up.

The emergency response is categorized in three levels that provide a framework for determining the severity of the event and the urgency of notifications and actions, Level 1 – Dam Incident, Level 2 – Dam Alert and Level 3 – Dam Emergency. The response levels are summarized in Figure 1.

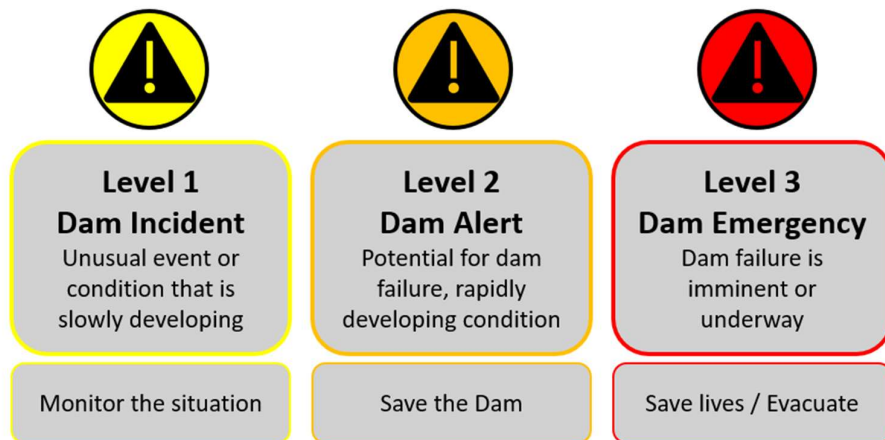


Figure 1. Emergency response levels

Emergency plans are intended to clearly define roles, responsibilities, and decision points within a dam emergency response. This includes identifying who is responsible for determining the emergency response level, initiating notifications, coordinating communications, and providing operational or technical support. Clear definition of these functions helps users understand

Strengthening Dam Emergency Preparedness: Lessons learnt from British Columbia, Canada

both individual responsibilities and how the overall response is intended to operate during an incident.

The effectiveness of an emergency response plan depends not only on its structure, but also on whether it is understood, maintained, and used effectively by those responsible for implementing it. This creates the need to look beyond the dam emergency plan and consider the broader preparedness activities that strengthen its practical use.

STRENGTHENING PREPAREDNESS BEYOND THE WRITTEN PLAN

A well-structured dam emergency plan (DEP) provides an essential foundation for emergency response, but the existence of a written plan alone does not ensure that a response will be effective in practice. Emergency preparedness is strengthened through a broader cycle of activities involving the people who will use the plan, including engagement, training, exercises, learning from emergency events, and ongoing plan review. This framework is illustrated in Figure 2.

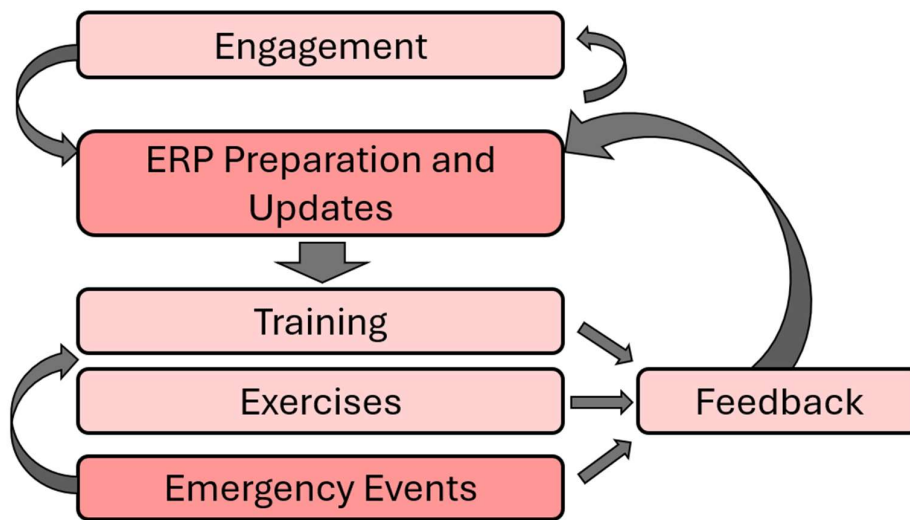


Figure 2. Framework for strengthening emergency preparedness

Key elements that contribute to strengthening emergency preparedness include:

- Engagement with stakeholders
- Training tailored to the individual role
- Emergency exercises using realistic scenarios
- Emergency events with post-incident learning
- DEP review and updates based on feedback

Engagement and training provide opportunities to build relationships, clarify expectations, and ensure that personnel understand their roles and responsibilities within the response. Exercises go further by testing decisions, communications, and actions through realistic scenarios. Although exercises are valuable for assessing how the response process is intended to function, real emergency events provide the most direct test of whether the plan can be implemented effectively under actual conditions. Minor emergency events can provide valuable learning opportunities by testing emergency procedures under real conditions without resulting in severe downstream consequences. Feedback from engagement, exercises, and post-incident debriefs can then be used to improve the DEP and strengthen future preparedness activities.

For dam owners and operators, this process is most effective when emergency preparedness is treated as an ongoing cycle of engagement, planning, training, exercising, and learning rather than as a one-time compliance task.

STAKEHOLDER ENGAGEMENT

A dam emergency response often does not occur solely within the dam owner's organisation and it often depends on multiple persons and organisations communicating effectively and responding in a coordinated manner. Stakeholder engagement is therefore essential as effective emergency response depends not only on the quality of the DEP, but also on whether the people and organisations involved understand their roles, communication pathways, and responsibilities before an emergency occurs.

It should include both internal personnel involved in response actions and external stakeholders who may receive notifications, provide support, or play a role in protecting downstream communities. Engaging a broader group of stakeholders helps ensure that emergency planning reflects how a response is likely to function in practice rather than how it is assumed to function on paper.

Some key areas of stakeholder engagement include:

- Clarifying roles and responsibilities before an incident
- Confirming notification pathways and contact points
- Identify unrealistic assumptions in the DEP
- Find communication gaps or conflicting expectations
- Building relationships and trust before they are needed in an emergency
- Making the plan more usable in practice

Strengthening Dam Emergency Preparedness: Lessons learnt from British Columbia, Canada

A key lesson from engagement is that it should begin early and continue over time. Roles, personnel, and organisational structures can change, and ongoing engagement helps maintain relationships, align expectations, and identify changes that may affect responsibilities, notifications, or response capability. Engagement also improves the quality of the DEP by revealing unclear responsibilities, communication gaps, practical constraints, and language that may not be easily understood by all users.

Follow-up is an important part of the engagement process. Issues raised during discussions should be recorded, reviewed, and, where appropriate, reflected in DEP updates, training, or future exercises. In this way, engagement supports not only stronger relationships, but also a more practical, usable, and widely understood emergency plan.

TRAINING

Training helps ensure that personnel understand the DEP, their role within it, and the overall response process before they are required to apply that knowledge during an exercise or emergency event. It provides the baseline understanding needed to support timely decisions, effective communication, and coordinated action.

All personnel identified in the DEP or notification chart should have an appropriate level of awareness of the plan and understand how they may be involved in a response. To account for the variety of roles included, training should be tailored to the level of involvement expected from each role. For some roles, this may require only a general understanding of the DEP structure, notification process, and expected communication pathways. For others, particularly those responsible for determining the emergency level, decision-making, notifications, or operational response, a more detailed understanding of actions, triggers, and response expectations is required.

Effective training helps build familiarity, confidence, and consistency in emergency response. By ensuring that personnel understand both the plan and their place within it, training provides the foundation for meaningful participation in exercises and for more effective response during real emergency events.

EXERCISES

Exercises are a key part of emergency preparedness as they test whether the DEP can be implemented effectively in practice and give responding staff the opportunity to walk through the steps that may occur during an emergency. Post-incident critiques often confirm that experience gained during exercises was the best way to prepare teams to respond effectively to an

emergency (Ready, 2026). Not only do exercises enhance knowledge of plans, they give an opportunity for responders to improve their performance during an emergency and identify opportunities for further learning.

Exercises help confirm whether roles and responsibilities are understood, whether communication systems and notification pathways function as intended, and whether critical information can be accessed quickly during an event.

A tabletop exercise provides a structured discussion based on a hypothetical scenario and allows participants to work through decisions, notifications, communications, and response actions. A tabletop exercise is the simplest way to test an emergency scenario, as it can be done online if there are challenges getting staff in one place and it requires the least preparation. The components of a tabletop exercise are shown in Figure 3.

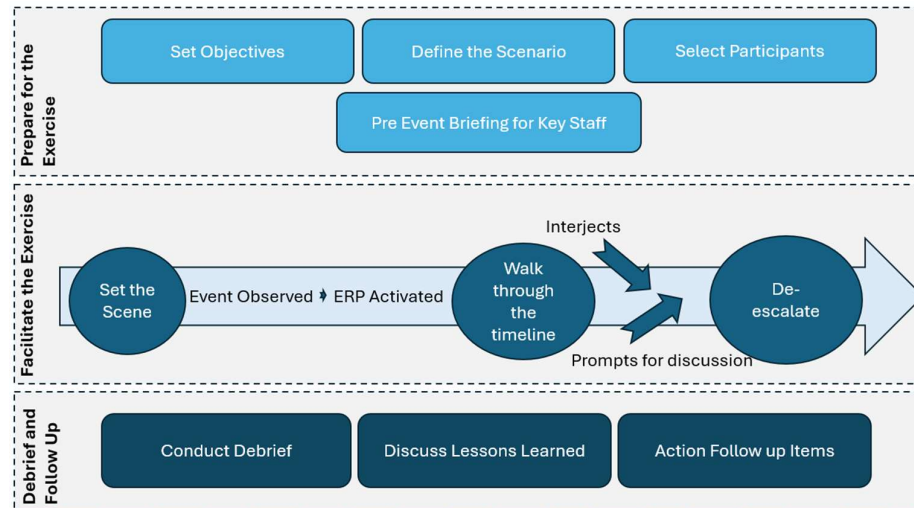


Figure 3. Components of an emergency tabletop exercise

A key lesson from emergency exercises is that scenarios should be realistic, relevant, and capable of escalation. Participants are more likely to engage meaningfully with a scenario that reflects credible operating conditions and plausible warning signs, such as seepage, slope instability, extreme, but realistic rainfall, or flood-related concerns. Presenting the scenario in stages, with injects that introduce new information or complications, can help test how participants respond to changing conditions, reassess classifications, and adapt their actions as the event develops.

Exercises should also be used to evaluate whether the DEP is workable as the DEP is followed during the exercise. Exercises can reveal unclear responsibilities, unrealistic expectations, roles with too much responsibility, gaps in communication pathways, and weaknesses in forms, notifications, or

Strengthening Dam Emergency Preparedness: Lessons learnt from British Columbia, Canada

decision logic. They may also identify where additional flexibility is needed in the plan, for example where staff availability, time of day, or concurrent demands could affect the response. In this way, exercises provide a practical bridge between the DEP and the realities of emergency implementation.

MINOR EMERGENCY EVENTS

Emergency events provide the most direct test of whether a DEP can be implemented effectively under real conditions. While exercises are valuable for simulating decision-making and response processes, real events introduce uncertainty, operational pressure, competing priorities, and communication demands that cannot be fully replicated in a planned setting. For this reason, minor incidents and developing emergency conditions can provide some of the most valuable opportunities for learning and improving preparedness.

Even where an event does not escalate to dam failure, activation of the DEP can reveal whether roles are clear, notifications are practical, communications are timely, and supporting information can be accessed quickly. Real events, that are the only way to really test how responders perform under pressure, may also highlight response actions that were not clearly described in the DEP, key information that was not easy to find, or show that certain assumptions in the plan do not reflect how the response actually unfolds in practice.

A key lesson from minor emergency events is that communication is often the point at which weaknesses become most apparent. As the number and diversity of affected downstream stakeholders increases, communication requirements become more complex. Weakness may include communication pathways that are not well established, and delays to information being distributed. Communication often needs to begin before all information is available and during a developing event, internal and external stakeholders may require early notification even where conditions are still being assessed. Pre-approved messaging, clear alert levels, and practical notification tools that allow for quick distribution of information can support timely communication and help build confidence that the situation is being actively managed.

Emergency events also reinforce the importance of DEP usability. During a real incident, personnel may be required to make decisions under stress, time pressure, and uncertainty, and should not be expected to interpret lengthy or complex documents in real time. The DEP should allow users to quickly identify required actions, distinguish key response steps, and find critical information such as notification charts and decision making tools.

Separating key information from background information, using checklists or bullet points and colour coding can support the usability of the DEP.

Following minor incidents, a structured post-incident debrief is crucial to identify what worked well, what barriers were encountered, and what updates are needed to improve plans, training, communication tools, and future exercises. In this way, minor emergency events support the same continuous improvement cycle as engagement and exercises, while providing the most direct evidence of how the response functions under actual conditions.

PUBLIC COMMUNICATION OF FLOOD MAPS

The CDA Guidelines for Public Safety Around Dams (CDA, 2011) state that the dam owner is responsible for managing the public safety risks caused by a dam, as far upstream and downstream as the owner has property rights. Beyond this property, the dam owner may have additional responsibility to assess specific locations where the hazards are known by the owner to result directly from the dam or its operation and to inform the public and other affected property owners of these hazards.

A public safety management system includes promotion of general public awareness of the dam and its associated hazards, as well as communication of any unique hazards associated with specific sites. General awareness is typically addressed through advertising strategies and education materials. Public safety communication can also support emergency preparedness by improving awareness of dam-related hazards and providing context for warning and evacuation planning.

Flood maps are an important component of a dam emergency plan because they identify areas that may be affected by dam failure and support emergency planning, warning, and evacuation arrangements. Their primary function is to support preparedness and response; however, in some cases they may also form part of a broader approach to public awareness of dam failure hazards. This may be particularly relevant for downstream communities located close to a dam, where warning and response times may be limited. Where appropriate, making this information available can help explain why emergency planning is necessary, support discussions about warning and evacuation arrangements, and provide useful context for those who may be affected by a dam incident.

Public communication of inundation information requires careful consideration. The purpose of the maps, together with their assumptions and limitations, should be clearly explained. Releasing maps without adequate context may create confusion, unnecessary alarm, or misplaced

Strengthening Dam Emergency Preparedness: Lessons learnt from British Columbia, Canada

confidence. Supplementary fact sheets or explanatory materials can be used to provide additional context, along with community outreach programs. Communication of flood maps is therefore best considered as part of a broader public engagement strategy rather than as a standalone disclosure exercise.

CONCLUSION

A dam emergency response plan is a necessary foundation for emergency response and a standardised dam emergency plan template, such as that used in British Columbia, gives dam owners a practical and accessible framework that can be readily adapted with site-specific information, improving consistency between plans while reducing the effort required to prepare them.

The existence of a written plan alone does not ensure preparedness. Practical experience from British Columbia demonstrates that effective emergency preparedness depends on a broader and ongoing process of engagement, training, exercising, and learning from real events. These activities help ensure that roles are understood, communication pathways function as intended, and emergency actions can be implemented effectively under real conditions.

Emergency preparedness should be treated as a continuous process of review, practice, and improvement rather than a one-time compliance task. Stakeholder engagement is essential to clarify responsibilities and ensure that emergency planning reflects how a response will work in practice. Training and realistic exercises build familiarity, support decision-making, and help identify weaknesses in the plan before an emergency occurs. Minor emergency events and post-incident debriefs then provide valuable opportunities to refine procedures, communication tools, and overall plan usability.

Minor emergency events are particularly valuable because they often reveal communication challenges that are less visible during planning or exercises, especially when information is incomplete, decisions are still evolving, and updates are needed for a wider group of stakeholders. These experiences reinforce the importance of engagement, training, exercises, and review as connected parts of preparedness. Engagement helps establish relationships and expectations before an event occurs, training builds familiarity with roles and message content, and exercises provide an opportunity to test communication under realistic conditions.

As the UK Reservoir Safety Reform Programme develops, these lessons may help dam owners move beyond the preparation of emergency plans toward

a more practical and resilient approach to preparedness. While emergency planning frameworks will differ between jurisdictions, the underlying principle is consistent: preparedness is stronger when plans are simple, usable, regularly tested, and supported by the people, relationships, and communication processes needed to implement them effectively. Public communication of flood maps may also support this process where it is carefully integrated into broader public outreach program.

REFERENCES

- British Columbia Ministry of Water, Land and Resource Stewardship (BCWLRS) (2024), *BC Dam Safety Program Annual Report 2023/24*, available at Dam Safety Program Annual Reports – gov.bc.ca (www.gov.bc.ca)
- British Columbia Ministry of Water, Land and Resource Stewardship (BCWLRS) (2016), *BC Dam Safety Program Annual Report 2016/17*, available at Dam Safety Program Annual Reports – gov.bc.ca (www.gov.bc.ca)
- British Columbia Ministry of Water, Land and Resource Stewardship (BCWLRS) (2023), *Water Sustainability Act, Dam Safety Regulation - B.C. Reg. 40/2016* (last amended February 10, 2023)
- British Columbia Ministry of Water, Land and Resource Stewardship (BCWLRS) (2023), *Instructions, Guide, and Templates for Preparing a Dam Emergency Plan (DEP) in British Columbia, Updated 2023-01*, available at Dam Safety Technical Resources – gov.bc.ca (www.gov.bc.ca)
- Canadian Dam Association (CDA). (2013). *Dam Safety Guidelines (2007, Revised 2013 Edition)*. Canadian Dam Association, Canada.
- Canadian Dam Association (CDA). (2011). *Guidelines for Public Safety Around Dams*. Canadian Dam Association, Canada.
- Ready, An official website of the U.S. Department of Homeland Security. Internet accessed May 18, 2026.
<https://www.ready.gov/business/training/testing-exercise/exercises>



The British Dam Society

The Impact of the Tous dam Incident on Modern Dam Safety Regulation

H. C. YARRITU-SANCHEZ, SICE, Reading, United Kingdom

J. FLEITZ, OFITECO, Madrid, Spain

C. SERRANO, Júcar River Basin Authority, Valencia, Spain

SYNOPSIS

Throughout history, changes in dam safety regulations have frequently been implemented in response to major dam failures. The incident at Tous in Spain (1982) is a clear example, marking a key milestone in the evolution of dam safety practices. It revealed shortcomings in operational decision-making and in the regulatory framework, emphasising the need to enhance asset management, clarify technical responsibilities, and improve the assessment of natural hazards.

The review of the Tous incident highlights how Spain has progressively reinforced its dam safety practices. This has resulted in adapting more consistent hydrological criteria, implementing real-time hydrometeorological sensor networks to better assess and predict inflows to reservoirs, enhanced emergency action plans, and stricter regulation of dam safety reviews, dam operation, and monitoring. Dam safety management has also shifted towards more structured and traceable documentation, supported by digital tools that enhance access to and use of inspection and monitoring data.

Although regulatory contexts differ from one country to another, the experience gained from Tous can help address similar gaps elsewhere. The lessons drawn illustrate how past failures continue to guide the modernisation of dam safety, supporting more resilient, preventive, and risk-informed management practices adapted to current challenges.

INTRODUCTION: MEDITERRANEAN CATCHMENTS AND DAM SAFETY

Mediterranean river basins are often described as systems of strong contrasts. Long periods of low or moderate flows can be suddenly interrupted by short-lived but intense rainfall events, producing flood responses that are disproportionate to the usual behaviour of the river. These episodes are not only the result of climatic variability, but also of the way Mediterranean catchments concentrate and route runoff during extreme conditions.

The Impact of the Tous dam Incident on Modern Dam Safety Regulation

A key factor lies in the morphology of the lower reaches, where many Mediterranean rivers flow through wide, low-gradient floodplains close to the coast. In these areas, channel slopes are typically very small (generally below 1–2‰), which limits the capacity of the river system to convey large discharges under high-flow conditions. Under normal conditions, these rivers are well adjusted to accommodate frequent high flows within the natural range of variability, but they are not capable of efficiently routing the exceptionally large discharges associated with rare storm events, leading to flooding of the adjacent floodplain.

These hydrological processes are closely linked to the atmospheric conditions typical of the Mediterranean. Certain synoptic configurations favour the advection of warm, moist air from the sea, sometimes combined with upper-level instability, leading to highly concentrated rainfall over relatively short periods. In extreme cases, recorded precipitation volumes over a catchment can reach very high values within hours, providing the input required to generate large flood waves. For the October 1982 event in the Júcar basin, the direct catchment upstream of Tous (6,780km²) received an average precipitation of 277mm, corresponding to a total rainfall volume of about 1,880hm³, illustrating the magnitude that such episodes can attain. A recent reminder of this behaviour was the DANA of 29 October 2024 in Valencia, when AEMET (State Meteorological Agency) reported 185mm in one hour and 621mm in six hours at the rain gauge located in Turís.

Exposure also plays a decisive role in shaping flood risk. Mediterranean floodplains are typically areas of high economic value and significant land use and development. Agricultural development, infrastructure and urban expansion have occupied zones that are naturally prone to inundation. This combination of hydrological behaviour and territorial pressure results in systems where relatively rare events can produce severe consequences.

Historically, responses to flood risk in these basins have relied heavily on structural measures, including channelisation works, levees and reservoirs for flow regulation. However, extreme events have repeatedly shown the limitations of these approaches when faced with highly exceptional discharges. In particular, rare floods can exceed design assumptions, challenge operational decision-making and expose gaps in both technical and institutional frameworks.

Mediterranean catchments provide a demanding environment for dam safety. The combination of highly variable hydrology, complex basin response and significant downstream exposure means that extreme flood events act as critical tests for infrastructure performance and management systems. The 1982 Tous event must be understood within this broader

framework, where uncommon but high-impact episodes play a central role in revealing vulnerabilities and driving changes in dam safety practice.

THE TOUS DAM AND THE 1982 EVENT

Dam characteristics and operation

Tous Dam is located in the lower reaches of the Júcar River Basin, approximately 15km upstream of the Mediterranean coast. It was conceived as a key flood control structure within a highly dynamic hydrological system, with the additional functions of river regulation, irrigation supply and water storage.



Figure 1. Tous Dam before the failure. Source: ICOLD (n.d.).

The dam was initially designed as an 80m high concrete structure, and construction started in 1958. However, during construction, two geological faults were identified in the riverbed, which led to the suspension of the works in 1964 and a subsequent redesign of the dam. After a 10-year redesign period, construction resumed in 1974, with the original scheme modified into a zoned rockfill embankment with a central clay core, integrated between the previously constructed concrete abutments. At completion, in 1978, the dam had a crest elevation of 98.5m with approximately 122 million m³ of reservoir capacity and included provisions to increase its height to an elevation of around 142m in a second phase that was never executed. The normal operation level was at 84m elevation, with a reservoir capacity of 51.5 million m³.

The dam had a central spillway equipped with three radial gates, designed to provide a total discharge capacity of about 7,000m³/s, corresponding to a flood event with a return period of approximately 500 years, in line with the design standards at the time.

The Impact of the Tous dam Incident on Modern Dam Safety Regulation

From an operational perspective, due to the use of a gated spillway, flood routing relied heavily on the controlled operation of the gates, strongly depending on the availability and reliability of mechanical and electrical systems. During the first years of operation, no significant incidents were reported, and the dam's behaviour was considered satisfactory under routine conditions.

This design approach reflects the engineering practice of the time, where flood safety was primarily ensured through structural capacity and operator-controlled discharge systems, with less emphasis on resilience, redundancy, real-time data or risk-informed decision-making frameworks, which are now more common approaches in modern dam safety practice.

The 1982 flood event

On October 20th and 21st, the Júcar River Basin experienced an exceptional storm episode that produced extreme rainfall over large areas of the inland catchment. The event is described as being driven by a cold, high-altitude depression interacting with warm, moisture-laden air, a combination that can trigger very intense rainfall in Mediterranean basins. Rainfall totals of around 500mm in 24 hours were recorded across wide areas.

Rainfall started late on 19 October and intensified on 20 October. The most intense period was between 08:00–14:00 on the 20th, with additional peaks later that afternoon. This timing explains the rapid rise in catchment runoff and the sustained loading of the river system during the same day.

The storm was not spatially limited. A large part of the basin received very high rainfall totals within the same time window. Isohyetal estimates indicate a central area exceeding 500mm, and when aggregated over the upstream catchment, the total rainfall volume is estimated at around 1,880hm³ upstream of Tous. This event was a very large water input distributed over a substantial portion of the basin.

The rainfall affected both the catchment upstream of Tous and the downstream basin. Reported averages are about 277mm over 6,780km² upstream and 226mm over 3,692km² downstream (roughly 1,880 and 834 million m³, respectively). This means there would have been widespread flooding and damage across the basin due to direct rainfall and river response downstream, even without any dam-break contribution.

The catchment response at Tous was extreme. Reconstructed inflows to the reservoir peaked at around 9,900–10,000m³/s. Just as important as the peak was the sheer volume of the event: the inflow hydrograph is estimated as at least 600 million m³, far greater than the storage available at normal operating level and several times the storage at crest level. As a result,

reservoir levels rose steadily through the day and the event quickly moved beyond the conditions for which the dam was designed.

Failure mechanism and contributing factors

The dam failed by overtopping during the afternoon of 20 October 1982. The failure resulted from a sequence of events in which loss of discharge control under rapidly rising inflows led to overtopping and progressive erosion.

Taken together, the incident is best understood as the combined effect of: (i) an extreme inflow event, previously explained, (ii) a gated spillway that could not be operated when it was most needed, and (iii) the inherent vulnerability of embankment dams to overtopping.

Therefore, the failure mechanism developed as follows:

1) Loss of gate operability and reduced discharge control

The first operational constraint was the loss of electrical supply. The grid was out of service due to the severe weather, and the on-site emergency power was not available: of the two diesel generators, one was under repair and the other could not be started. Moreover, the manual attempts to raise the gates were unsuccessful, so the gates remained closed during the event.

2) Rapid reservoir rise

During October 20th, reservoir levels rose steeply as inflows increased. Recorded observations show the level climbing from 80 meters (a.s.l) in the morning to around 99.5 meters (a.s.l) by the early evening. In parallel, water is reported to have begun passing over the spillway gates while they remained closed, and the reservoir continued to rise thereafter.

3) Overtopping onset and progressive erosion

The transition from severe event to failure occurred once the reservoir approached crest level. Overtopping is reported to have started in the late afternoon, although the precise timing varies between reconstructions (approximately 16:50–17:30).

During the overtopping phase, the reservoir is reported to have reached a maximum level of about 1m above crest (≈ 99.5 m.a.s.l.). At this point the situation was no longer governed by outlet capacity and reservoir routing, erosion progressed rapidly and the breach formation accelerated.

The Impact of the Tous dam Incident on Modern Dam Safety Regulation

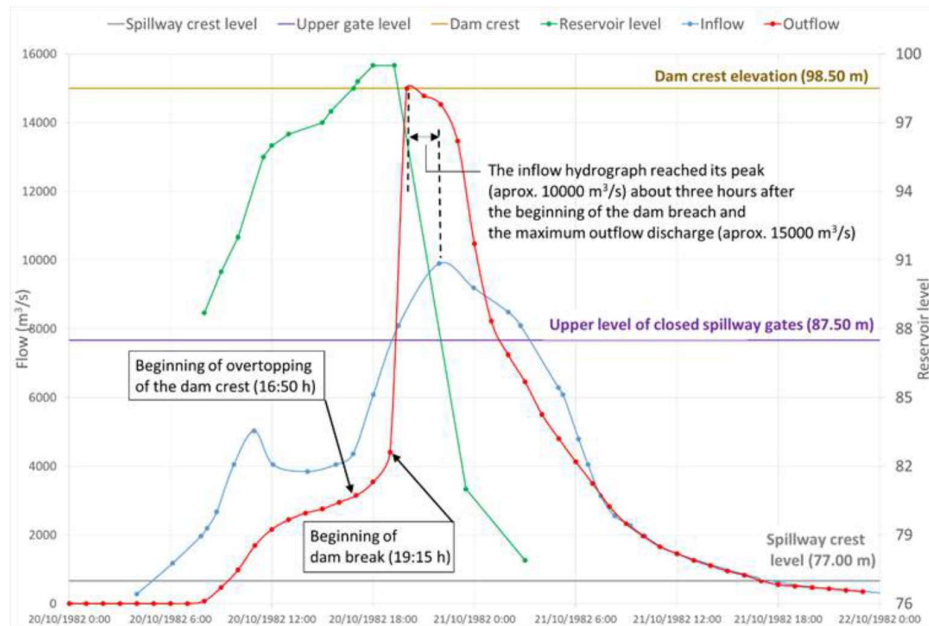


Figure 2. Inflow and outflow hydrographs and reservoir level evolution during October 20 and 21 at Tous Dam. Source: ICOLD (n.d.).

4) Breach formation and collapse timing

A significant part of the dam is considered to have failed at 19:15, close to peak reservoir level. The dam was overtopped and then washed out within roughly one hour. Erosion removed a large part of the shoulders and the central rockfill. Additionally, overtopping also led to the loss of the leftmost gate and its supporting structure.



Figure 3. Tous Dam after failure in 1982. Source: Fleitz, J. (2013).

The breach outflow wave itself is reconstructed as reaching a peak on the order of $15,000 \text{ m}^3/\text{s}$. Then, the reservoir continued to draw down rapidly, falling to around 81 m.a.s.l. by 23:30 on 20 October.

Downstream, the flood caused widespread inundation across the lower Júcar basin, affecting numerous towns and agricultural areas.

IMPACT ON SPANISH DAM SAFETY REGULATION

The failure had a strong impact on Spanish society and on the dam engineering community, particularly among responsible dam engineers. Within the professional debate, Tous became a reference case that exposed the limitations of design-based flood safety and highlighted the importance of operational reliability during extreme events.

A further driver was the legal and institutional aspects. The case led to a long litigation process, in which engineers involved in operation were condemned by Spanish legal authorities. In parallel, the engineering assessment highlighted that the flood event exceeded the design basis and that, even with fully operable and open gates, the available routing capacity would not have prevented reservoir levels rising beyond crest level.

This dual outcome reinforced the need for clearer operational rules, defined responsibilities, and better protection for dam engineers through formalised procedures and documented decision-making.

From design compliance to dam safety program

Before Tous, dam safety was largely framed around demonstrating that each dam met the required design and operating criteria on paper. In practice, this meant showing that the structure and its discharge facilities complied with the prescribed standards and could route the design event under the assumptions used at the time.

After Tous, the approach shifted. The incident is described as triggering the start of a **Safety Program for Large Dams** (initially focused on state-owned dams). Under this approach, safety was no longer treated only as project-by-project compliance, but as an ongoing programme of review, supervision and management, supported by documented operating procedures, routine surveillance and inspections, emergency planning, and follow-up actions to ensure that measures were implemented and maintained over time.

Key milestones

The regulatory pathway that followed can be summarised through a series of milestones that progressively formalised both technical requirements and operational preparedness:

- **1982 – Safety Programme for large dams.** The failure was the starting point of a formal “Safety Program for Large Dams” first thought for

The Impact of the Tous dam Incident on Modern Dam Safety Regulation

state-owned dams. This marks the transition towards structured oversight at portfolio level, rather than an asset-by-asset design check.

- **1992 – Preparation of new dam safety standards.** The General Directorate of Hydraulic Works decided to prepare new standards for dam safety, commissioning this task to the Commission of Large Dams Regulations. These standards aim to update the 1967 "Instruction for the Project, Construction and Operation of Large Dams". This effort led to a set of regulations and eight Technical Guidelines developed by the Spanish National Committee on Large Dams (SPANCOLD).

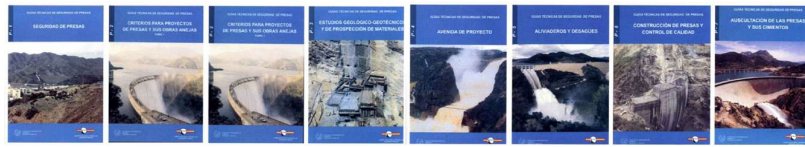


Figure 4. SPANCOLD Guidelines. Source: Source: Fleitz, J. (2013).

- **1995 – Basic Guideline of Civil Protection Planning at Flooding Risk.** A national civil protection directive was published, explicitly including emergency planning for the risk of failure or malfunctioning of dams. It was approved on December 9, 1994, and published on January 31, 1995. This is a key shift: dam safety requirements were no longer only “infrastructure-focused”, but also formally linked to downstream emergency planning. The 1995 directive laid the groundwork for modern flood management, eventually leading to updated national plans in 2011 and updated directives in 2026.
- **1996 – Technical Regulation on Dam and Reservoir Safety.** A dedicated technical regulation was approved, reshaping dam safety management in Spain by integrating civil protection requirements introduced in 1995, establishing a risk-based classification system, and requiring emergency action plans. It set out a more comprehensive framework covering both design and operation, with greater emphasis on reducing downstream consequences. Key aspects are:
 - Hazard-Based Classification: Dams are classified according to the potential consequences for human life, essential services, and environmental or material assets. Categories: A, B or C.
 - Emergency Action Plans: Defines requirements for emergency management plans (EAPs) for high-hazard dams.
 - Surveillance: Mandates formal monitoring, inspection, and maintenance programs led by qualified personnel.

- Safety Reviews: A systematic and periodic review of existing dams, including flood routing capacity and structural integrity.
- **2008 – Common obligations and responsibilities for all owners.** A Royal Decree established a common set of obligations for all dam owners and explicitly stated that dam safety management should be risk-based. This marked a further shift away from requirements applying only to certain assets or owners towards a more consistent, system-wide framework. Key obligations introduced in 2008 and related legislation include:
 - Classification and Risk Analysis: Owners must classify dams according to height, storage capacity, and potential downstream risk.
 - Technical Surveillance Programme: Owners must implement continuous monitoring and surveillance programmes designed by qualified technicians.
 - Emergency Action Plan (EAP): Owners of high-hazard dams must prepare and maintain an EAP.
 - Maintenance and Repair: Owners are responsible for maintaining structural integrity and carrying out routine repairs to prevent failure.
 - Environmental Liability: Under Law 26/2007 and the related 2008 regulations, owners must provide financial security for potential environmental damage.

A further consequence of the Tous failure was the implementation of Automatic Hydrological Information System (SAIH) across Spanish river basins. These systems provided real-time hydrometeorological information on basin conditions, strengthening flood forecasting and supporting earlier operational decision-making during flood events.

Taken together, these milestones show how Tous acted as a catalyst for moving from a largely design-event-driven approach to a more integrated framework combining design, operation, surveillance and civil protection.

THE DAM TODAY (CURRENT SAFETY FRAMEWORK AND OPERATION)

The Tous Dam currently in operation was built on the same site after the 1982 failure, incorporating major changes in both design and operation in response to the deficiencies identified during the event.

The Impact of the Tous dam Incident on Modern Dam Safety Regulation



Figure 5. The new Tous Dam. Source: ICOLD (n.d.).

From a hydraulic perspective, the most significant change is the spillway concept. The original dam relied on a gated spillway with a capacity of about $7,000\text{m}^3/\text{s}$ (500-year return period), while the new dam is equipped with an ungated spillway with a much higher capacity, around $20,000\text{m}^3/\text{s}$ (10,000 - year return period). This represents a clear shift away from reliance on gate operability under extreme conditions.

The reservoir has also been substantially enlarged due to heightening the dam. The new scheme provides significantly higher storage and flood routing capacity compared to the original configuration, which had a capacity of around 122 million m^3 at crest level, while the new reservoir has approximately 792 million m^3 of capacity at a crest elevation of 162.5m a.s.l.

Additionally, the current operation incorporates explicit flood management margins by adopting seasonal freeboards. The reservoir is operated at lower levels during the flood season, maintaining available storage for attenuation before releases are required.

In this configuration, reservoir regulation during flood events is not based on spillway control. Instead, discharge is initially managed through the outlet works. The new Tous dam includes both intermediate and low-level outlets, and it is reported that the intermediate outlet alone is capable of releasing flows on the order of $1,200\text{m}^3/\text{s}$ before the spillway becomes active. The ungated spillway therefore acts as a passive safety feature, providing additional discharge capacity once reservoir levels exceed its crest.

Beyond flood control, the scheme operates as part of a broader multipurpose system within the Júcar basin. The reservoir contributes to water supply and irrigation and forms part of a wider regulated system that also considers energy production and integrated water management.

These design and operational features are embedded within a broader safety framework that now includes defined operating rules, regular inspections, and emergency planning, supported by real-time hydrometeorological information.

LESSONS LEARNT

The Tous failure is widely interpreted not as the result of a single deficiency, but as a system-level failure in which design assumptions, operational capability and emergency response were all challenged simultaneously. The lessons derived from the event have therefore been reflected not only in engineering practice, but also in regulatory and institutional frameworks.

- The event demonstrated that compliance with design flood criteria alone does not ensure safety. As a result, since the 1990s, **dam safety frameworks increasingly evolved towards system-based approaches**, incorporating not only hydraulic design but also operation, monitoring and emergency preparedness within regulatory requirements.
- The loss of discharge control due to failure of the power supply highlighted that the **availability of critical equipment must be ensured under extreme conditions**. This led to a stronger regulatory focus on ensuring that essential systems remain functional under extreme conditions, which place emphasis on operating procedures, system reliability and redundancy as well as safety reviews. In the new Tous dam, this meant a much more robust electrical supply arrangement, including multiple transformation centres (each of them with redundant backup generators), and switching arrangements that allow the outlet works to be supplied from different centres.
- The Tous case contributed to a reassessment of gated spillways in extreme events. While not formalised as a single requirement, this is reflected both in **design practice** (e.g. ungated spillways in new schemes) and in **operational guidance**, reducing dependence on gate operability during critical conditions.
- The importance of maintaining available storage before extreme events led to the incorporation of **operational rules within dam safety frameworks**, including seasonal reservoir management. This reflects the integration of dam operation into broader flood risk management policies, recognising reservoir operation as a key element of flood safety.
- The case highlighted that outlet works are essential components of the discharge system. Modern practice requires that they provide

The Impact of the Tous dam Incident on Modern Dam Safety Regulation

sufficient and reliable discharge capacity before spillway activation, forming part of the overall safety system. Their role is implicitly reinforced through safety regulations and operational guidelines.

- Finally, Tous reinforced the **need to extend dam safety beyond the structure itself**. This is reflected in the **incorporation of real-time hydrometeorological monitoring, early warning systems and Emergency Action Plans (EAPs) into regulatory frameworks**. This was formalised through the 1995 Civil Protection Directive for flood risk and its integration into the 1996 Technical Regulation on dam and reservoir safety, linking dam operation with downstream risk management and civil protection policies.

REFERENCES

- Alcrudo F and Mulet J (2007). Description of the Tous Dam break case study. *Journal of Hydraulic Research*, **45**(1), pp. 45–57.
- ASDSO (n.d.). Tous Dam (Spain, 1982). *Dam Failures and Lessons Learned*. Available at: <https://damfailures.org/case-study/tous-dam-spain-1982/>
- Carmona P and Ruiz J M (2000). Las inundaciones de los ríos Júcar y Turia. *Serie Geográfica*, **9**, pp. 49–69.
- Fleitz, J. (2013). *Dam safety management in Spain*. Almaty, Kazakhstan.
- ICOLD (n.d.). *Tous dam failure due to overtopping. ICOLD Bulletin on Dam Surveillance – Lessons learnt from case histories*. International Commission on Large Dams, Paris, France
- Témez, J.R. and Mateos, C. (1993). Hidrogramas de entrada a Tous: Datos para un juicio crítico. *Revista de Obras Públicas*, No. 3319, Year 140, March 1993.
- Utrillas Serrano, J.L. (2013). *La presa de Tous. Ingeniería, seguridad y desarrollo en la Ribera del Júcar*. Confederación Hidrográfica del Júcar.



The British Dam Society

Design and Foundation Improvements at Yusufeli Dam to Accommodate Complex Geotechnical Conditions

Q H W SHAW, Concrete Dams Specialist, ARQ Consulting Engineers, U.K.

J KLEBERGER, Senior Geotechnical Consultant, iC Consulenten, Austria

D AYDOGAN, Deputy General Manager, State Hydraulic Works, DSI, Turkey

SYNOPSIS

Yusufeli Dam is a 275m high, double curvature concrete arch dam, which was constructed for the Turkish State Hydraulic Works on the Çoruh River in north-eastern Turkey between 2014 and 2022. The dam is the highest dam in Turkey and the 8th highest in the world, comprising approximately 4 million m³ of mass concrete and retaining an impoundment of 2.13 billion m³. Concrete placement for the dam structure saw a world record average rate exceeding 140 000 m³/month.

Investigations early during construction revealed substantially more complex geotechnical conditions than assumed during the Final Design, which required modifications to the dam design and a programme of improvement of the foundation rock mass.

In this paper, the authors describe the geotechnical conditions encountered, the measures developed to improve the rock mass, the associated dam design modifications and the methods applied to verify the improvements achieved. The correlation of predicted and measured behaviour during the staged impoundment verification studies and the accuracy of state-of-the-art FE modelling are demonstrated.

BACKGROUND

Yusufeli Dam impounds a reservoir of 2.13 billion m³ on the Çoruh River, which forms the storage for a 558MW underground power station that generates an average of 1900GWh of electrical energy per annum. The dam is a 275m high double-curvature conventional mass concrete arch structure.

The Turkish State Hydraulic Works, DSI, awarded the contract for the construction of the dam and power works with site operations commencing in early 2014. For DSI projects, the contractor takes responsibility for the construction investigations and design, although any necessary changes must be motivated for the approval of DSI before implementation. iC Consulenten from Austria was appointed for the geotechnical investigations, analyses and designs and a team comprising ARQ Consulting Engineers of

Design and Foundation Improvements at Yusufeli Dam to Accommodate Complex Geotechnical Conditions

South Africa and local consultants Su Yapi was appointed for the dam design. While ARQ led the dam design team, Su Yapi provided support and was fully responsible for all aspects of the power works.

INTRODUCTION

Early during the geotechnical investigations, a picture of more complex geotechnical conditions across the dam site started to emerge. With a uniform and competent foundation crossed by a single fault assumed in the Final Design, a network of more than 20 faults of influence was identified during investigations, together with various issues related to unfavourable joint orientation and apparent rock mass relaxation.

In this paper, the authors describe how a comprehensive rock mass model was developed in conjunction with design modifications to the dam and a structured programme of rock mass improvement.

The techniques used to confirm the adequacy of the rock mass improvement are recounted, while the staged impoundment and the dam performance verification processes are described.

INVESTIGATIONS AND ROCK MASS MODEL DEVELOPMENT

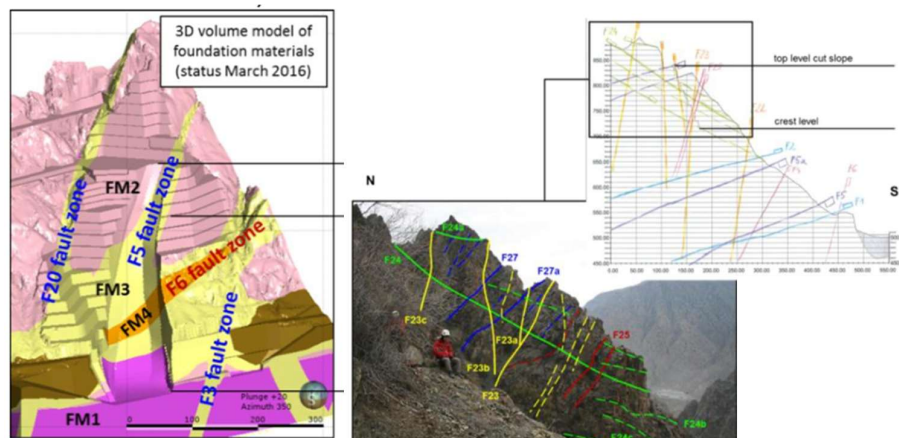


Figure 1: Rock mass model & joint mapping on left abutment

Geotechnical investigations at the Yusufeli site were initiated in 2014 and continued until the foundation rock mass model was finalised at the end of 2018. Investigations included over 2.3km of exploratory tunnels and galleries, 12.5km of boreholes, over 500 dilatometer tests, over 2km of sonic logging, over 5km of seismic refraction, 1200m of down-hole and cross-hole seismic survey, 44 plate load jacking tests and four large-scale shear tests. Additionally, all rock exposed in access tunnels and on excavated terraces was comprehensively mapped.

Through this process, a comprehensive and complex 3-dimensional rock mass model was developed using the Leapfrog software and the foundation rock mass was classified into six different rock types, each with different density, deformation and shear strength properties. Establishing the parameter sets for each of the six rock types considered rock material, jointing characteristics and degree of confinement/relaxation. The maximum elastic modulus determined was 13GPa and much of the foundation rock mass that had been assumed as 15GPa was demonstrated to be as low as 3.5GPa, as illustrated in *Figure 2*.

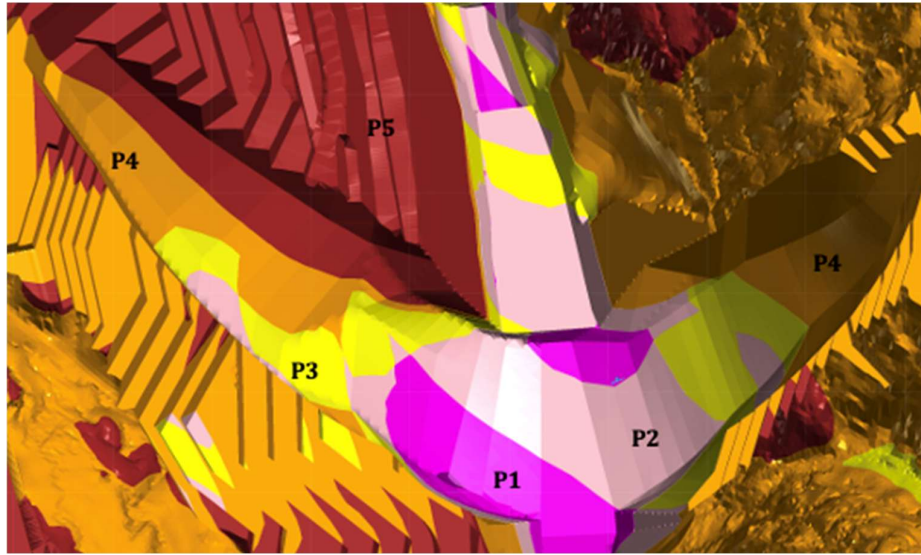


Figure 2: Final foundation rock mass model

The property sets for each of the distinct zones of the rock mass were defined as illustrated in *Table 1*.

Table 1: Geotechnical properties of foundation rock types

Rock Type	Density (kg/m ³)	Friction Angle (°)	UCS (MPa)	Def. Mod (GPa)	El. Mod (GPa)	Poisson's ratio	Confinement
P0	2700	42	31	10.0	13.0	0.27	Cf1
P1	2700	40	26	8.5	11.0	0.29	Cf1
P2	2700	37	16	6.5	8.5	0.30	Cf1-2
P3	2600	35	11.5	5.0	6.0	0.35	Cf2
P4	2500	32	7.2	3.0	3.5	0.43	Cf3
P5	2500	32	4.3	1.7	2.0	0.45	Cf4

Design and Foundation Improvements at Yusufeli Dam to Accommodate Complex Geotechnical Conditions

Critically, ongoing relaxation on the upper left abutment was measured between 2016 and 2017 and unfavourable joint orientations gave rise to the presence of prominent, potential sliding planes and consequential rock wedge stability issues.

EARLY DESIGN ANALYSES & SOLUTION DEVELOPMENT

In tandem with the rock mass model development, structural analyses of the dam and foundation system were performed to test the sensitivity of the arch structure to the various investigation findings and to develop appropriate mitigatory solutions. The distinct rock type volumes defined in the Leapfrog model were imported directly into the MIDAS NX FEA software, with only minor modifications to ensure compatibility.

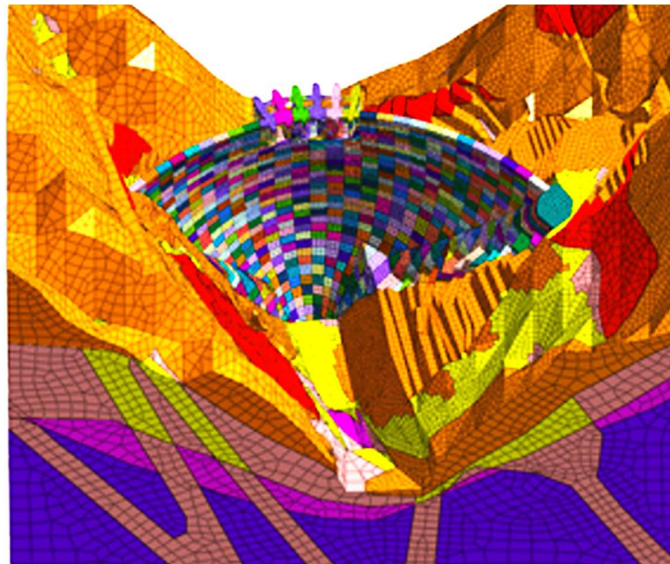


Figure 3: FE model of dam and foundation

Early analyses demonstrated high tensions in a direction parallel to the axis along the dam/foundation contact, which were a direct result of the deformations associated with the low foundation deformation moduli. It was clear that specific measures would be required, both to accommodate larger deformations and to limit the extent of the foundation deformation under load. Investigations to establish the extent of improvement that could be achieved with grouting were initiated, while modifications of the design to provide a transition between the stiff concrete of the dam and the more flexible foundation rock mass were developed. In addition, the relaxation of the rock mass on the upper left abutment required deeper excavations, which again necessitated a modification of the structural design.

As the investigations progressed and the design modifications were developed, the designers were required to identify solutions within the constraint of a confined footprint. With the excavations proceeding, the platforms under construction and three cranes in manufacture, design modifications to the dam for the developing geotechnical conditions could not be allowed to spill out beyond the reach of the cable cranes.

Due to the steepness of the valley abutments, the cut slopes needed to rise to heights of 150m above the dam crest. This situation was made more critical by the increased depth of excavation on the left abutment. In places the design required a slope steepness of 1 horizontal in 10 vertical. With natural slopes in some places already at limits of stability, a fixed InSar interferometric radar slope surface monitoring system was installed. To ensure safety during construction, readings were taken over the left abutment at 5-minute intervals.

Upper left abutment

Analyses were also performed to evaluate the various wedges created through a number of unfavourably oriented joint sets at the top of the left abutment. All related joint sets were included in the FE model, applying (Mohr Coulomb) interface elements to represent the respective joint surfaces, and analyses were performed under normal load to establish the critical joint sets. On the basis of this information, the critical wedges were established, as illustrated in Figure 4, and analysed for stability.

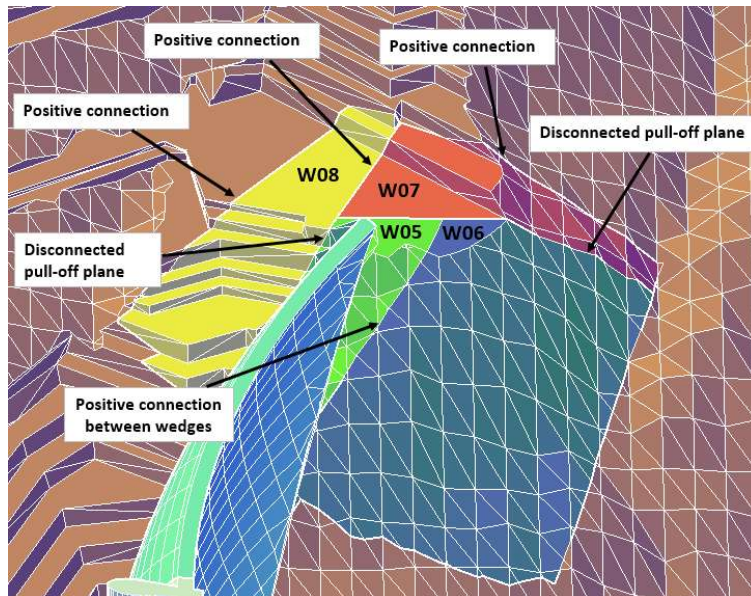


Figure 4: Critical wedges on upper left abutment

Design and Foundation Improvements at Yusufeli Dam to Accommodate Complex Geotechnical Conditions

These analyses demonstrated insufficient factors of safety, with or without the dam, and it was established that an infill structure in the gully immediately downstream of the dam was required to provide additional kinematic restraint (see *Figure 5* and *Figure 12*).

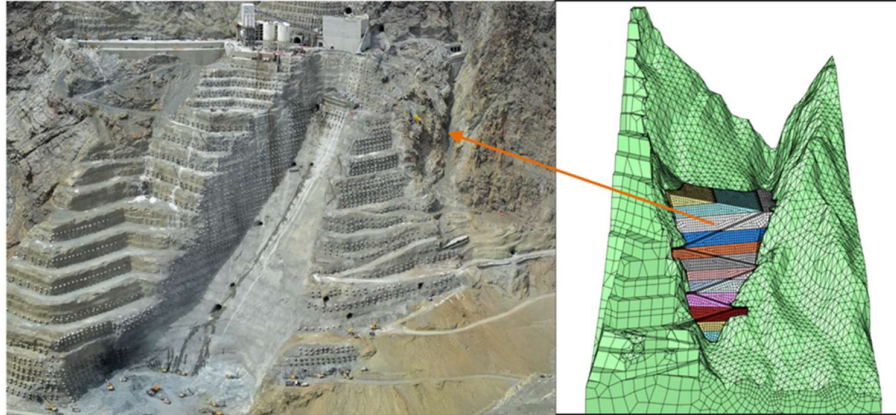


Figure 5: Location of infill structure and model of structure constructed in Hardfill

The final infill structure was 126m in height and comprised approximately 265 000m³ of Hardfill.

“Cushion concrete”

Analyses further demonstrated that the application of a transition concrete between the dam and the foundation rock mass was effective in controlling the problem tensions in the dam structure. For this purpose, a concrete, that was henceforth termed “cushion concrete”, with a 0.4F_{cu} secant elastic modulus of 19GPa was targeted. Various means to effectively halve the elastic modulus of the concrete were investigated at the DSI laboratories, including using bentonite, an increased sand content and a very high fly ash content, but the optimal solution was found in an entrained air content of 11%. This solution proved to be particularly effective, due to an increased level of creep under sustained loading.

Where the dam traverses a location at which two faults intersect, as can be seen on *Figure 1*, it was found to be necessary to further enhance the tensile strain capacity of the cushion concrete. This was achieved using polymer fibre reinforcement in the rock contact zone.

Cushion concrete was used on the right flank from the base to approximately 3/4 dam height. On the left abutment, cushion concrete was integrated into, and beneath a concrete abutment, which essentially proved an effective means to accommodate deeper excavations without significantly impacting structural symmetry. Additionally, this solution contained construction

within the reach of the cable cranes, while retaining the aesthetics of symmetry.

The cushion on the right flank and the combined cushion and concrete abutment on the left flank are illustrated in Figure 6. The figure presents a simple illustration for information, and it should be noted that the cushion and abutment were constructed simultaneously with the dam.

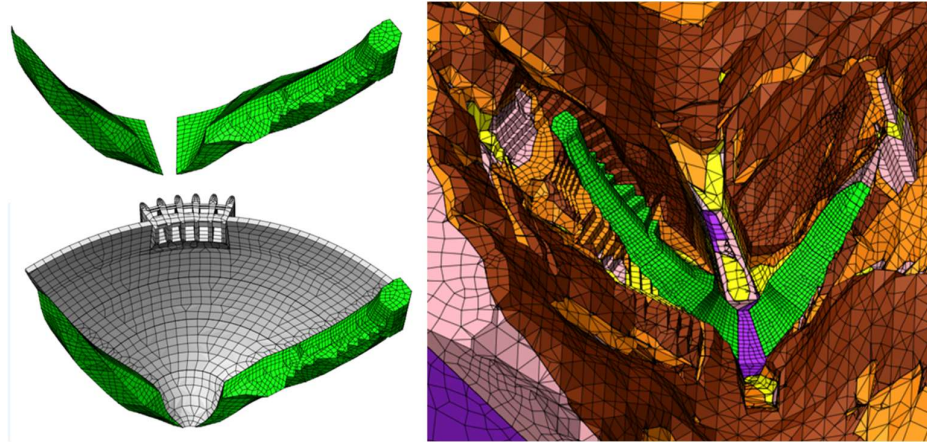


Figure 6: Cushion on right flank & combined cushion/concrete abutment on the left

The design modifications made to meet the challenges associated with the more complex geotechnical conditions increased the total concrete volume of the dam from approximately 2.3 million m³ to almost 4 million m³.

FOUNDATION IMPROVEMENT

General

The primary subject of this paper is the process through which the foundations at Yusufeli Dam were improved, how the improvements were verified and how measured behaviour was correlated with modelled predictions to confirm dam safety, both during impoundment and early operation.

Investigation and Testing

The rock mass relaxation that occurred between 2016 and 2017 demonstrated the way a reduction in seismic P wave velocities developed as internal joints within the rock mass opened. Test panels were established in various galleries and adits where access could be gained to the different rock types, from P1 to P5. Plate-load tests were performed before any improvement, together with seismic refraction and dilatometer testing. The testing was repeated after grouting and the data successfully demonstrated that seismic refraction, particularly using a cross-hole approach,

Design and Foundation Improvements at Yusufeli Dam to Accommodate Complex Geotechnical Conditions

represented a reliable means to verify the achievement of the desired improvement (Pöschl & Frei, 2023).

The initial testing unsurprisingly revealed that very little improvement could be gained in the better rock (P0 to P2), obviously due to the fact that there were no open joints to take grout. The lowest quality fault gouge material was also demonstrated to be relatively impermeable, consequently taking little grout and not indicating a reliable improvement. Realistically, it was accordingly only the P3 and P4 rock mass types that indicated reliable improvement through grouting, with a commensurate increase in indicated E value of at least 2 to 2.5GPa (Kieffer et.al. 2019).

With a knowledge of the achievable improvement, the next step was to establish the extent of the foundation volume requiring improvement to assure satisfactory structural performance. Grouting from the temporary galleries included in the dam toe against the foundation, a series of analyses were performed with the FE model to develop an optimal grouting bulb. The rock mass volume grouted is illustrated in *Figure 7*.

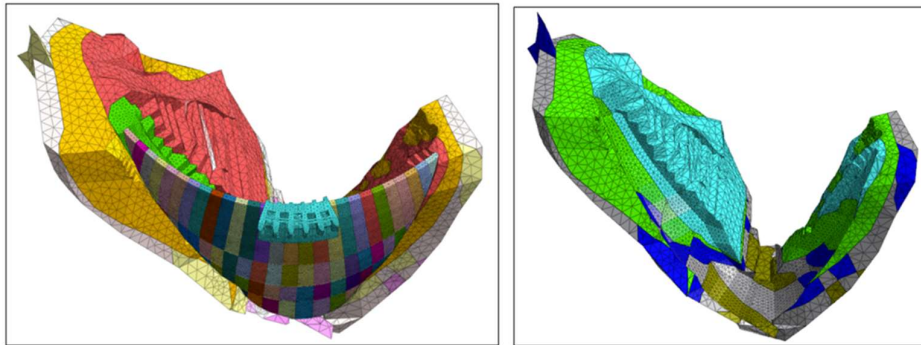


Figure 7: Extent of foundation improved by grouting - with and without dam

Improvement and Verification

The process of rock mass improvement was completed in five phases, from the bottom upwards, testing the rock mass before grouting and verifying the improvement achieved afterwards, phase by phase. In the occasional location where testing indicated the required improvement was not achieved, the contractor was required to repeat the process, re-grouting and re-testing until the applicable criteria were exceeded.

The improved rock mass was also evaluated where possible through its in-situ response under the mass of the dam during construction by comparing displacements and extensometer readings on the prototype dam with equivalent figures extracted from the structural model.

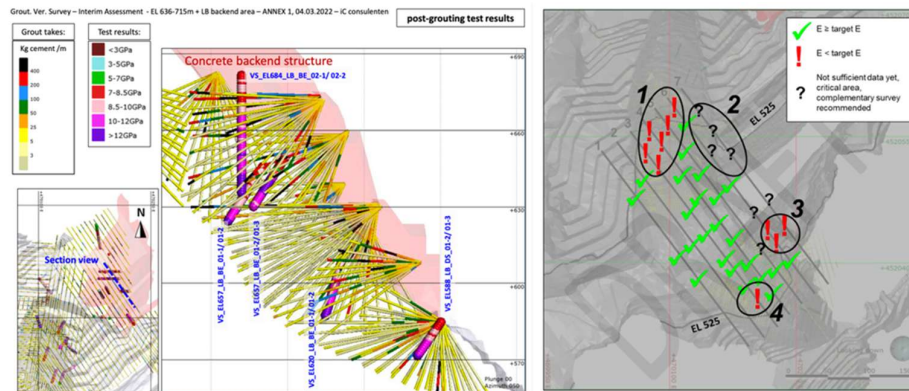


Figure 8: Illustration of improvement grouting in phases + process of verification

PERFORMANCE VALIDATION

General

First impoundment at Yusufeli Dam was initiated at the end of November 2022 and programmed over six stages, with a hold point of 30 days between each stage. Except for the first stage, when the water level needed to rise to the elevation of the bottom outlet, the rate of rise was capped at 1m per day. It was envisaged that the first impoundment process would have a duration of 16 months. At each of the hold points, the dam behaviour would be monitored closely and continuation to the next stage would be contingent on acceptable observed performance. At several of the hold points, the intention was to verify measured behaviour against that predicted through modelling. Because the behaviour of an arch dam is almost as much related to temperature, as to water load, it is essential to accurately model the temperature state in a structural analysis. Accordingly, the modelling process was extended in duration by the need to extract temperatures measured within the dam and apply these to develop a realistic temperature distribution within the structural model.

Impoundment Verification

The impoundment reached 98% maximum water depth over a period of nine months, but was drawn down to use storage for generation during the winter of 2023/24 and maximum normal water level was finally reached at the end of June 2024. After being held at the level of the bottom outlet for a longer period than initially intended, because of the late relocation of various infrastructure within the basin, the water level was raised as rapidly as safely possible until inflows diminished in late summer 2023.

The excellent behaviour of the dam throughout first impoundment allowed the intended hold periods to be reduced substantially. At five of the stages,

Design and Foundation Improvements at Yusufeli Dam to Accommodate Complex Geotechnical Conditions

the measured behaviour was compared with the predicted behaviour, with the exceptional correlation allowing the following stage to be initiated with complete confidence.

The impounded water level reached in 2023 represented 95% of the full water load on the structure and drawdown during the subsequent winter for generation reduced loading to approximately 50% full loading (see *Figure 9*).

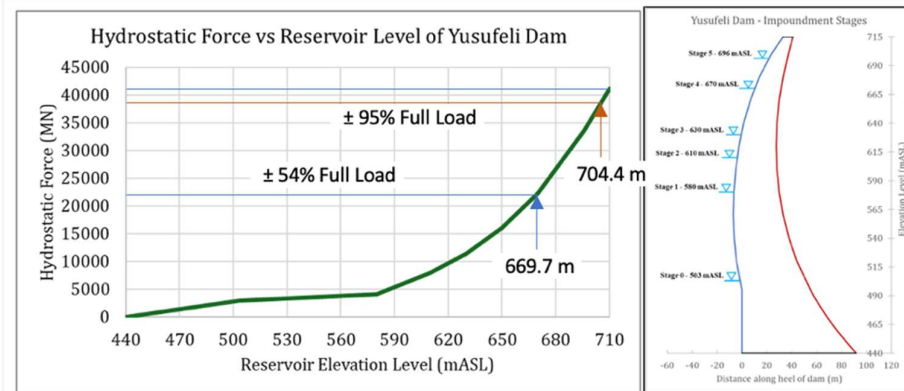


Figure 9: Illustration of total hydrostatic load against reservoir water level

Reaching maximum normal water level in mid-2024, the impoundment was again drawn down during winter 24/25, although to a slightly lesser extent but for a longer period, again reaching full in July 2025. By mid-2025, the dam had essentially experienced two full annual operational cycles.

Comparing Measured and Predicted Performance

Measured and predicted behaviour was compared on all installed instrumentation, with the most useful measurements being displacements; on the installed hanging and inverted pendulums and through geodetic survey. Twelve fixed displacement beacons were installed on the downstream side of Yusufeli Dam, and these are automatically monitored by laser from a fixed station.

The history of displacements measured on beacons B01 to B12 from impoundment to August 2025 is illustrated in *Figure 10*. The indicated response clearly reflects the two seasonal/operational cycles.

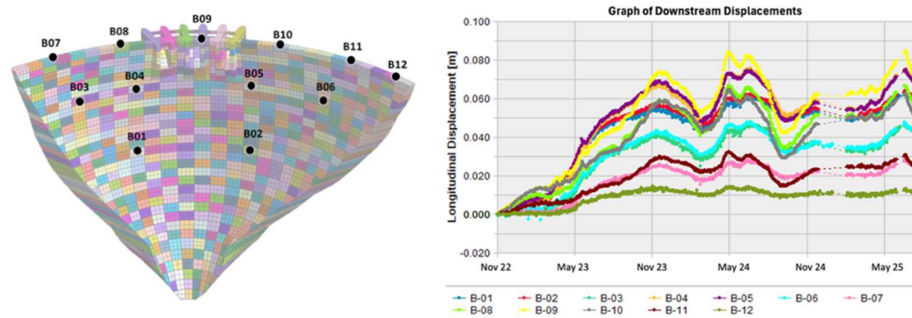


Figure 10: Geodetic survey beacon and readings over first 30 months of impoundment

Through all stages of the first impoundment for which structural analyses were run, the measured displacements on the geodetic beacons reflected very closely the predicted values, as illustrated in Figure 11.

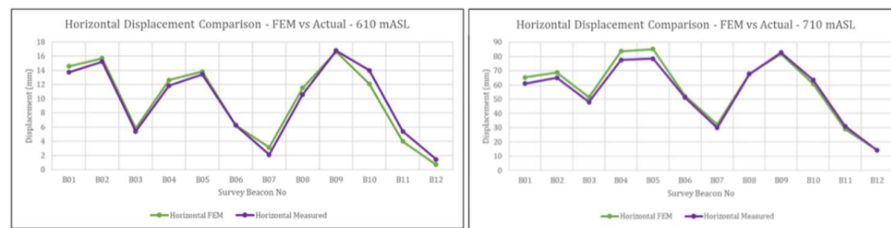


Figure 11: Measured and predicted beacon displacements for stages 2 and 6

Very importantly, no plastic deformation was observed at any stage during the first impoundment, or since. Displacements have uniformly reflected an elastic response, increasing with increasing water load, but remaining constant at each hold point. Furthermore, no plastic, irreversible deformation was apparent through the two refilling cycles after first filling.

It should be noted that the close correlation between the measured and predicted behaviour was not only a consequence of effective foundation treatment, but also the accuracy of the structural modelling, using the MIDAS FEA system. Interface elements with Mohr Coulomb, non-linear, properties between the dam structure and the foundation, for example, allowed an accurate replication of the arch response under load.

DISCUSSION

Yusufeli Dam is the fifth highest concrete arch dam in the world. The four higher arch dams are all located in China and were constructed between 2010 and 2021. It would accordingly be true to say that the technology of super, or ultra-high arch dams is relatively young. Chinese experience with super-high arch dams (Fu et al, 2021) indicates that displacements continue

Design and Foundation Improvements at Yusufeli Dam to Accommodate Complex Geotechnical Conditions

to increase significantly during the first several years after first impoundment and increases of the order of 30%-50% relative to a pure elastic response have been observed. A similar behaviour was observed at the 186m high Katse Dam in Lesotho, with plastic, irreversible displacements of the crest increasing almost linearly to approximately 22%, over the first six years of operation. While a number of causes have been hypothesised, this phenomenon likely occurs due to a combination of factors, including thermal adjustments, the release of built-in stresses, creep, and rock mass/foundation adjustments under saturation.

While it would be quite surprising if no increase in displacement is measured within the first five years at Yusufeli Dam, the fact that none has been seen to date instils confidence in the effectiveness of the design and construction.

CONCLUSIONS

A comprehensive 3D rock mass model and an accurate and representative FE structural analysis model allowed appropriate and innovative solutions to be developed and implemented to mitigate the more complex geotechnical conditions encountered at Yusufeli Dam.

Excellent correlation between the predicted and measured behaviour of the dam/foundation system corroborated the good performance of the dam, the effectiveness of the improvement measures implemented and the accuracy of the mathematical analysis model. This situation allows for an unusual level of confidence in the observed dam behaviour and, most importantly, a commensurate level of confidence in the safety of the dam.



Figure 12: Yusufeli Dam on first impoundment

REFERENCES

- Kieffer S D, Dreesse T, Weil J and Kleberger J (2019). Tools for optimizing rock mass grouting. In: *Geomechanics and Tunnelling* **12(2)**, pp. 121 -128.
- Fu Y, Tan Y, Hu Y, Liu C, Pei L and Wang Y (2021). Study on deformation and force of a super high arch dam during impoundment period in southwest China. *Proceedings. 7th International Conference on Hydraulic and Civil Engineering & Smart Water Conservancy and Intelligent Disaster Reduction Forum*.
- Pöschl I and Frei W (2023). The use of dynamic moduli logging in large-scale geotechnical modelling. In *15th ISRM Congress 2023 and 72nd Geomechanics Colloquium*, (2023), pp. 830 - 835, ISBN 978-3-9503898-3-8



The British Dam Society

Dam Safety Regulatory Compliance Challenges for Small Undertakers in Spain

M. P. ELÍAS, OFITECO, Madrid, Spain

H. C. YARRITU-SANCHEZ, SICE, Reading, United Kingdom

J. C. DE CEA, MITECO, Madrid, Spain

I. GARCÍA, MITECO, Madrid, Spain

SYNOPSIS

Since 2021, Spain has enforced new legislation on the safety of dams and their reservoirs, through Royal Decree 264/2021 of April 14, which establishes the Technical Safety Standards for Dams and Reservoirs.

The application of several provisions of this decree can be complex, particularly for older structures or those with unique characteristics. Even so, the new regulatory framework has increased owners' awareness of infrastructure safety and has encouraged greater investment aimed at improving the condition and reliability of dams throughout the country.

SPANISH LEGISLATION SINCE 1967

Royal Decree 264/2021 of April 14 establishes the Technical Safety Standards for Dams and Reservoirs in Spain, defining the fundamental principles governing their design, construction, commissioning, operation, and decommissioning. Its purpose is to set minimum safety requirements applicable throughout the entire life cycle of dams and reservoirs, ensuring the protection of people, property, and the environment. These requirements apply to all dams located within Spanish territory, regardless of ownership or location.

Prior to the approval of R.D. 264/2021, dam safety in Spain was regulated under a fragmented framework. The 1967 Large Dams Instruction applied only to dams classified as "large" at that time, while the 1996 Technical Regulation extended safety obligations to large dams and to those classified under risk categories A and B, subject to specific administrative conditions. In parallel, the obligation to classify dams by risk had already been introduced in the 1994 Basic Guideline for Civil Protection Planning against Flood Risk. This coexistence of regulations resulted in uneven application of safety criteria.

In order to unify dam safety requirements and clearly define the responsibilities of competent authorities, Chapter VII of the Public Hydraulic Domain Regulation was developed under Royal Decree 9/2008, proposing

Dam Safety Regulatory Compliance Challenges for Small Undertakers in Spain

the adoption of a single regulatory framework based on three Technical Safety Standards. With the entry into force of R.D. 264/2021, these standards became the sole binding legal reference, repealing previous regulations. While public authorities have progressively ensured compliance across state-owned dams, a significant gap remains among small private owners, many of whom are still unaware of the current legal obligations and the risks associated with non-compliance.

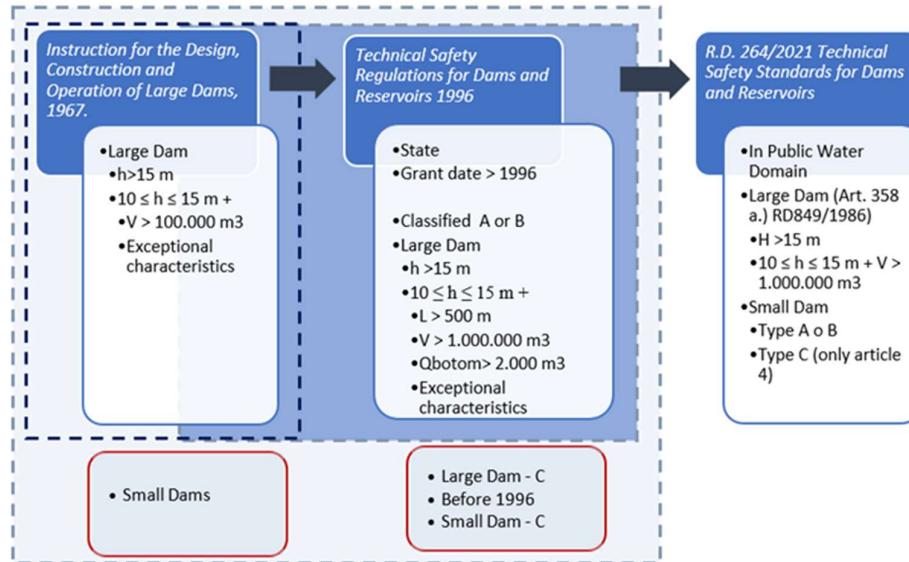


Figure 1. Developments in Spanish regulations.

STRUCTURE OF R.D. 264/2021

Royal Decree 264/2021 approves the Technical Safety Standards for Dams and Reservoirs, comprising three standards whose applicability depends on the specific stage of the dam's life cycle. These standards are included in the Royal Decree as three annexes:

- Annex I: Technical Safety Standard for the classification of dams and the development and implementation of emergency plans for dams and reservoirs.
- Annex II: Technical Safety Standard for the design, construction, commissioning, and filling of reservoirs.
- Annex III: Technical Safety Standard for operation, safety reviews, and decommissioning of dams.

Dams already in operation at the time of the publication of Royal Decree 264/2021 are not automatically required to comply with the design, construction, and commissioning provisions set out in Annex II.

Nevertheless, most of the criteria contained therein were already included in previous Technical Standards and were widely known and applied in practice, even though they were not legally binding. With the entry into force of R.D. 264/2021, these criteria have become mandatory within the regulatory framework.

Furthermore, Annex III reintroduces the relevance of Annex II criteria for existing dams by establishing them as a reference framework for safety assessment. When a dam does not meet any of these criteria, the owner must carry out a detailed analysis of the deviations and, where necessary, propose corrective measures to improve safety, incorporating these analyses into the Safety Reviews. Consequently, regulatory compliance largely depends on the owner's technical capacity to justify the dam's safety before the competent authority through rigorous analysis and appropriate mitigation measures.

CONSEQUENCES OF THE PUBLICATION OF ROYAL DECREE 264/2021

The immediate consequence of the new regulation was the need to conduct all necessary actions to ensure compliance across dams and reservoirs. These actions primarily fall under two categories:

1. Updating and improving safety documentation, including classifications, operating rules, emergency plans, safety reports, and operation review reports, and accelerating their administrative processing.
2. Assessing the structural and operational safety of dams and ancillary works according to the requirements of the Royal Decree.

Over the past five years, all Spanish dams higher than 5m - approximately 3,000 structures - have been visited regardless of ownership. During these inspections, contact was established with each owner, their Technical Archive was reviewed, and an on-site assessment was conducted to evaluate the safety conditions of each infrastructure. Given the large number of dams and the diversity of contexts, numerous distinctive challenges have emerged, as well as potential solutions to improve safety when necessary.

Moreover, a significant proportion of Spanish dams are more than 50 years old, predating even the Technical Standards supporting the 1996 Regulation. This raises pertinent questions: How does the new regulation affect such structures? Do they meet the criteria in Annex II? Below are some of the most frequently recurring issues.

Annex II (NTS2 C2 SII – 4) requires considering the effects of climate change when updating hydrological studies. Current analyses rely on studies

Dam Safety Regulatory Compliance Challenges for Small Undertakers in Spain

conducted by CEDEX (2021) and climate simulations from EURO-CORDEX and AEMET (2016). Climate change introduces significant uncertainty, sometimes increasing peak flows or modifying hydrographs toward more “flashy” behaviour. A substantial rise in hydrological studies incorporating climate change is expected as documentation is updated.

MAIN SAFETY DEFICIENCIES IDENTIFIED IN SOME DAMS OWNED BY SMALL PRIVATE HOLDERS.

Because of the dam inspections, significant deficiencies in documentation, structural safety, and operational conditions have been brought to light.

Issues Related to Structural Safety

Many dams suffer from at least inadequate maintenance. A considerable number of embankment dams have been constructed without a supporting technical design, through the accumulation of impermeable material until reaching the required elevation. As a result, dams have been identified presenting slope instability mechanisms, such as slip circles, as well as extensive vegetation on slopes, including trees whose curved trunks indicate ongoing or past sliding processes.



Figure 2. Slope instability and dense vegetation on the downstream slope

In masonry dams, it has also been common to identify significant uncontrolled cracking, lack of proper seating at abutments, and relative movement between adjacent blocks. Some structures are located at high elevations where freeze-thaw action is frequent. Observed issues include material detachment, the presence of cyanobacteria, and evidence suggestive of alkali-aggregate reactions, such as crack mapping patterns or calcareous efflorescence. In certain concrete dams, the growth of trees on the downstream face has also been observed.



Figure 3. Inadequate abutment support and longitudinal cracking in the spillway



Figure 4. Damage to the dam face caused by freeze–thaw action and tree growth

Issues Related to Hydraulic Safety

Seepage and Ponding

In dams lacking adequate maintenance, the appearance of seepage over time is common. The problem lies not only in the origin of the seepage itself, but also in the absence of proper monitoring, both in terms of flow rate and water characteristics, such as color and potential material transport. Clayey seepage has been identified in some cases, which may indicate the loss of impermeable material from the dam body.

Dam Safety Regulatory Compliance Challenges for Small Undertakers in Spain



Figure 5. Clayey seepage

The presence of water at the downstream toe of the dam has also been observed, the origin of which may be associated with seepage through the foundation.



Figure 6. Seepage through the foundation

Spillways

Under the former 1967 Instruction, the term “Maximum Flood” referred to the 500-year return period flood, which determined the design of spillways and other flood evacuation elements.

Annex II of the new Technical Standards (NTS2 C2 SII – 8) defines the return periods of design and extreme floods based on dam category (A, B, or C) and typology (concrete or embankment). Required return periods range: Design Flood: 100 to 1,000 years; Extreme Flood: 500 to 10,000 years

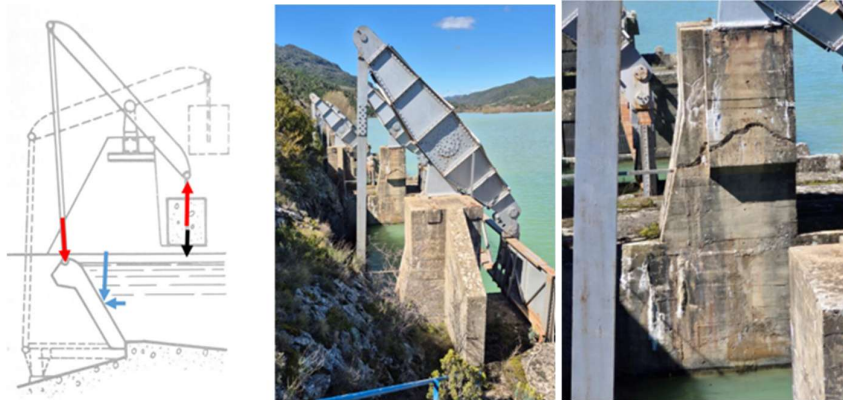


Figure 7. Inaccessible automatic spillway

Unsurprisingly, updates to Operating Rules and Safety Reviews frequently identify non-compliance with safety margins due to these increased flood levels, in some cases even resulting in overtopping under extreme flood scenarios. Hydroelectric dams are particularly affected, as operational strategies often involve maintaining high reservoir levels.

Consequently, owners must evaluate these changes and adopt measures to ensure compliance with hydrological and hydraulic safety requirements.

In other situations, the problems have arisen due to insufficient forethought during the design phase, such as the discharge of the spillway directly onto bottom outlets or onto the dam abutments themselves or structures without erosion protection..



Figure 8. V Spillway discharge over abutments and access staircase to the bottom outlet

It goes without saying that the presence of “foreign objects” within the spillway prevents the proper evacuation of the design flood flows, and such obstructions must be removed in all cases.

Dam Safety Regulatory Compliance Challenges for Small Undertakers in Spain



Figure 9. Erosion of the spillway and presence of a “foreign object” within the spillway

Existence and operation of bottom outlet

The existence of a functional bottom outlet is essential for safe reservoir operation, as it allows controlled drainage at any water level and plays a key role in flood management. Nevertheless, many reservoirs lack an operational bottom outlet due to poor maintenance, extreme age, or because one was never built. In several cases, the existing outlets are more than 75 years old or have remained closed and unused for decades.

Among dam outlet works, bottom outlets are often the most uncertain element for operators. Frequently, there is no reliable discharge–valve opening relationship, which complicates drawdown planning and flood response. Other common issues include the presence of only a single conduit—despite later classification as a Category A or B dam—and long response times, particularly for manually operated systems or those without automatic level monitoring.

For dams upgraded to Category A or B after construction, especially embankment dams, outlet works are often inadequate to evacuate floods associated with current design return periods. This deficiency may result from inoperative outlets, effective discharges lower than design values, or significant operational restrictions. Therefore, regulatory compliance under modern safety standards is frequently compromised.

In some cases, dam owners have tried to compensate by using water intakes as auxiliary drains to increase discharge capacity. However, Annex III of R.D. 264/2021 explicitly prohibits this practice. Exceptionally, it may be accepted with strong technical justification and strict operational rules governing reservoir levels, drawdown rates, downstream discharge limits, and

emergency procedures, although such solutions are generally assigned lower operational priority due to safety concerns.



Figure 10. 18th century dam and reservoir facilities

A particular challenge is posed by historic dams, such as an 18th-century masonry dam built under King Felipe V, now classified as Category A and protected as cultural heritage. Lacking a conventional bottom outlet, this structure highlights the difficulty of applying modern regulations to pre-engineering designs. In such cases, a comprehensive technical and historical assessment may allow legacy elements, originally conceived as water intakes, to be reinterpreted as functional equivalents to bottom outlets, achieving safety objectives while preserving historical integrity.

Diversion Dams

In dams designed to store water transferred from other basins, intakes are sometimes used as bottom outlets. These structures typically impound a small local catchment while receiving water from a nearby stream with greater flow, usually through pumping systems. Given the limited size of the contributing basins, design and extreme flood events are generally small, and spillways are sized with sufficient safety margins to meet regulatory requirements. As a result, such dams commonly include two conduits operating as bottom outlets, equipped with dual gates and redundant systems, thereby complying with established safety standards.

The main difficulty arises when one conduit is required to operate simultaneously as a bottom outlet, intake, and pumped inflow conduit. Although the presence of two dedicated bottom outlets may be unnecessary due to the low flood magnitude and adequate spillway capacity, strict and well-defined operating rules become essential. These rules must address bidirectional flow control, reservoir drawdown procedures, and coordination during flood events. Furthermore, since these conduits are often oversized for drainage purposes, any justification for their use as

Dam Safety Regulatory Compliance Challenges for Small Undertakers in Spain

bottom outlets must ensure—through operational and discharge analyses—that downstream releases do not exceed the limited conveyance capacity of the receiving channel.

The Pyrenees lakes

“Ibones” are natural glacial lakes whose storage capacity was later increased by dams built at their outlets. In most cases, the bottom outlet lies above the original lake intake, meaning that only the volume created by the dam—not the natural lake volume—can be evacuated during floods. Because these dams were constructed before the 1996 Regulation, their design parameters and extreme flood capacities often fail to meet current standards, making overtopping during extreme events relatively common. Consequently, flood management is effectively limited to artificially created storage.

The Mar Lago dam illustrates this issue: although its total capacity is 13.64hm^3 , only about 1.36hm^3 (around 10%) corresponds to dam-created storage. Lowering the normal operating level could reduce this volume below 1hm^3 , allowing reclassification as a small dam and exemption from most R.D. 264/2021 requirements, albeit with economic impacts for the operator. Alternatively, compliance could be achieved through costly structural upgrades, such as raising the dam. Given the small catchments, short concentration times, and high-altitude conditions with frequent snow accumulation, it may be technically reasonable to allow control overtopping, provided it is supported by adequate structural checks and robust technical justification, thereby ensuring compliance without imposing unnecessary financial burdens.

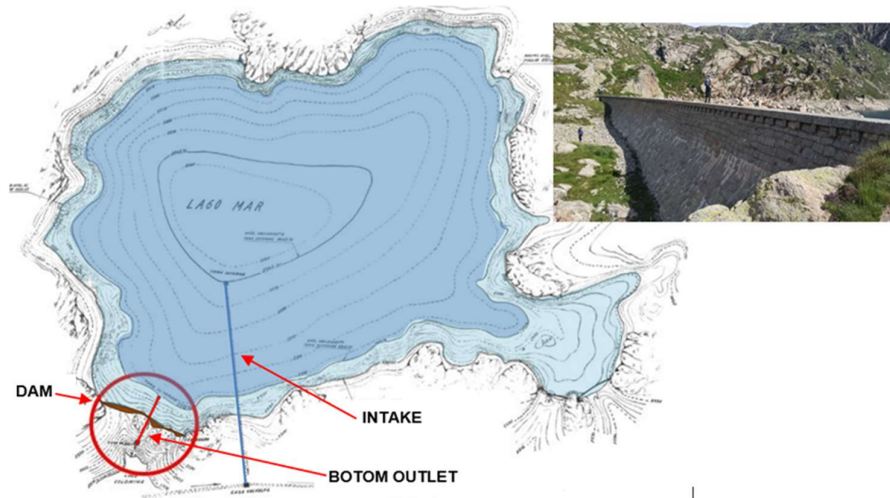


Figure 11. Ibón and Mar Lago Dam (Lérida).

Instrumentation

Instrumentation is a fundamental component of dam safety, both during construction—where monitoring can reveal early signs of potential issues—and throughout operation, as dams must continuously demonstrate safe behaviour through reliable data. Spanish regulations have progressively strengthened this requirement, from the minimum surveillance measures established in 1969, through the obligations introduced in the 1996 Regulation, to Royal Decree 264/2021, which mandates a formal Monitoring Plan and defines minimum instrumentation systems as essential elements for safety assessment and emergency management.

Although many dams in Spain have adequate monitoring systems in place, greater emphasis is now required on improving their quality, maintenance, and systematic data analysis. High-quality instrumentation has already enabled the detection of anomalous behavior in several cases. However, a significant number of dams, particularly smaller ones, still suffer from deficient, abandoned, or nonexistent monitoring systems, often due to limited technical capacity of their owners. This highlights the need for enhanced technical support and increased awareness of regulatory obligations.



Figure 12. Examples of non-operational monitoring.

CONCLUSIONS

Royal Decree 264/2021 and its associated Technical Safety Standards were developed to integrate all existing dams in Spain under a unified regulatory framework. Their implementation often requires not only infrastructural adaptations and updates to safety documentation, but also a change in the operational approach of dam owners and operators, placing a stronger emphasis on safety culture and regulatory compliance.

Although the legislation has highlighted deficiencies that, in many cases, should have been addressed earlier - since it largely reaffirms design criteria already defined in previous Technical Standards - it has acted as a catalyst for a sector that had remained relatively stagnant for years. Inspections

Dam Safety Regulatory Compliance Challenges for Small Undertakers in Spain

carried out to date indicate that substantial work is still required, either through upgrading existing infrastructure or by technically justifying the safety of dams that do not meet Annex II criteria but may nevertheless be considered safe based on their observed performance.

REFERENCES

- Spain (2021). Real Decreto 264/2021, de 13 de abril, por el que se aprueban las normas técnicas de seguridad para las presas y sus embalses. Available at: <https://www.boe.es/eli/es/rd/2021/04/13/264/con>
- ICOLD (2024) *Annual Meeting. Dams for People, Water and Environment and Development*. International Commission on Large Dams, New Delhi.



The British Dam Society

Proportionality and reservoir reform

CW SCOTT, Independent

AJ BROWN, Jacobs

S BUTLER, Canel & Rivers Trust

P FARNELL, Severn Trent Water

JR FOSTER, Mott MacDonald

M HAYWARD, Stillwater Associates

T JUDGE, Scottish Water

SYNOPSIS

The Independent Reservoir Safety Review Report prepared by Professor David Balmforth and published in March 2021 recommended that the principle of proportionality should apply to reservoir safety with greater effort (and investment) applied to reservoirs at the higher end of the hazard range and less at the lower end. The Environment Agency commissioned consultants to conduct research and develop the new hazard classification system and improved safety management system. This work was completed in early 2025 and the output published in July 2025. The research is understood to form the basis for the reservoir safety reform proposals being developed by Defra and the Welsh Government. At the time of writing the details of those proposals are not in the public domain.

The research proposes a new classification system and a range of safety management practices associated with the different hazard classes. In accordance with UK Government Practices, a Regulatory Impact Assessment (RIA) was carried out to analyse the potential costs, benefits, and risks of proposed new regulations (or changes to existing ones) before they are implemented, to endeavour to determine whether policies are necessary, effective, and proportionate.

This paper is produced by the members of the PEC (Panel Engineers Committee of the ICE) and reviews the proposals in the research reports and the assumptions made in the RIA to consider whether the proposals are proportionate and will result in an increase in the safety of reservoirs. It will include consideration of a range of credible assumptions, the implication for the cost to individual reservoir owners for high and low hazard dams, and the magnitude of potential benefits to society, relative to other risks to society.

Proportionality and reservoir reform

INTRODUCTION

The Independent Reservoir Safety Review Report prepared by Professor David Balmforth and published in March 2021 recommended that the principle of proportionality should apply to reservoir safety. In discussing the principle of proportionality Balmforth states:

“All reservoirs currently designated as “high-risk” are subject to the same periodic supervision and inspection by qualified reservoir engineers. The current threshold that is used to designate “high risk” is set relatively low and results in a considerable range of hazard posed by high-risk reservoirs. It is not appropriate for the safety at all high-risk reservoirs to be managed in the same way. The principle of proportionality should apply, with greater effort (and investment) in reservoirs at the higher end of the hazard range and less at the lower end.”

Balmforth goes on to recommend that high-risk reservoirs should be divided into three classes and that the threshold between classes should be determined by the Environment Agency in consultation with Defra, the ICE and the BDS.

Balmforth also recommended that Government should review the current threshold for designating high-risk reservoirs.

The Environment Agency commissioned research to identify a method for determining thresholds and the requirements, termed safety management practices, that would apply to reservoirs across the different classes. The work was done through 2024 and completed in early 2025. The research reports were published in July 2025. Work has continued by others to flesh out the proposals in those research documents but as of the date of writing this paper, those research reports remain the only substantive information in the public domain available for comment and discussion.

The research was one of a suite of activities undertaken by Defra to implement the recommendations in the Balmforth report.

SUMMARY OF PROPOSALS

Hazard Classification

The research proposes a hazard classification with four classes: three broadly aligned with “high-risk” and the fourth with not high-risk reservoirs. It is proposed that the hazard of reservoirs is assessed using a combination of three factors:

- *Threat to life*: Represented by ASLL (Average Societal Loss of Life), this metric is derived from Reservoir Flood Mapping (RFM) and is a measure of the potential societal impact of a dam failure.

- *Hazard potential from energy release in dam failure:* The energy a dam may release in the event of a failure is represented by a combination of dam height (H) and reservoir volume (V) using the following formula:

$$H^2 \times \sqrt{V}$$

- *Potential downstream damage:* Represented by property damages, another consequence metric derived from RFM.

The proposed means of combining these factors to give a hazard classification is summarised in Figure 1.

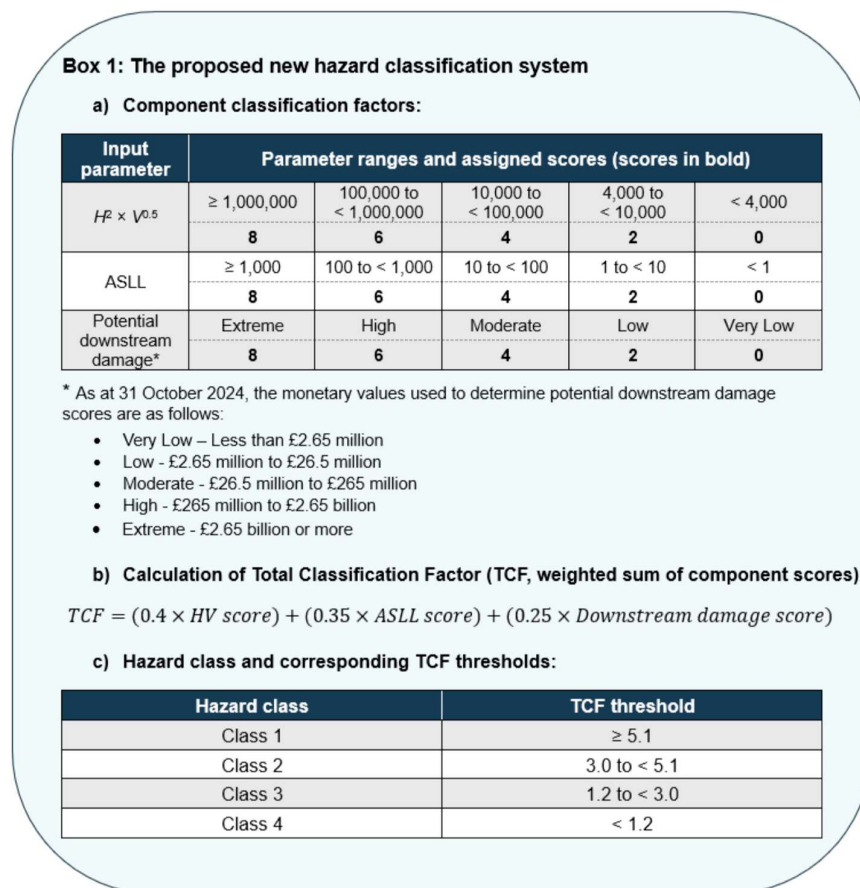


Figure 1 Proposed classification in Final Report of Hazard Classification Research

Safety Management Practices

The research proposes safety management practices that vary across the different hazard classes. The proposed safety management practices are summarised in the table below.

Proportionality and reservoir reform

Table 1. Safety Management Practices

Safety Management Practice	New or existing	Description
Reservoir Safety Case (RSC)	New	Sets out the key safety elements of the reservoir's design and operation, used to inform the RSMP.
Risk Assessment	Existing	Completed as part of the RSC to identify failure modes and risks. Tier of risk assessment required depends on class of reservoir.
Reservoir Safety Management Plan (RSMP)	New	Sets out how the owner will operate and manage their reservoir safely, informed by the RSC and includes other SMPs.
Surveillance	Existing	Regular review of the reservoir assets to check condition and notice and act on any changes.
Monitoring	Existing	Reading and interpretation of instrumentation and monitoring equipment. Frequency and nature of monitoring required depends on the class of reservoir.
Maintenance	Existing	Plan for implementation of activities to ensure that reservoir assets remain in good condition and key elements remain functional.
Operating Procedures	Existing	Documentation of operating procedures for key structures including the dam, inlet structures, outlet works, overflow and spillway structures.
Record Keeping	Existing	Requirements for maintaining records in a specified location, likely to include mandatory records such as the Prescribed Form of Record (PFR) and additional information about the reservoir.
Emergency Planning	Existing	Emergency plan to outline the areas at risk and actions to be taken to mitigate an emergency event at the reservoir.
Panel Engineer Supervision	Existing	Appointment of a Supervising Engineer to supervise the reservoir as required by the current Reservoirs Act. In addition, the SE will assess the compliance of the owner and operator with the procedures set out in the RSMP and report in the Annual Statement.

Safety Management Practice	New or existing	Description
Periodic Safety Review (PSR)	Existing	Similar approach to the current inspection and reporting requirements of Section 10 of the Reservoirs Act. PSR considers the current RSC Case (including risk assessment) and RSMP and advises of any updates that are required. PSR includes an assessment of the reservoir design and performance against latest standards and guidance.
Incident Reporting	Existing	Requirement for all owners and operators to report to the Enforcement Authority any incident which could have, or did, affect the safety of any large raised reservoir and has required emergency measures.
QCE Oversight & Certification		A role similar to existing Construction Engineer and QCE roles but simplified with a consistent approach across the activities and lifecycle stages.

It is important to note that, while many of these activities are already being undertaken, they are not necessarily documented in the manner that will be required by the proposed changes in legislation.

It is envisaged that the requirements of certain of the safety management practices would vary depending on the hazard class of the reservoir. The proposed application of these safety management practices across hazard class is shown for reservoirs in service in Table 2.

Regulatory Impact Assessment

As part of the hazard classification project a cost benefit assessment was done. The assessment considered the costs of the new requirements over a 50-year period and compared them to the benefits in reduced risk of failure over the same period. The assessment was done on a Present Value basis of the 50 years of costs and benefits.

The costs of the various safety management practices were estimated and summed to determine the cost of the changes. Details of the costs are outlined in the research report.

The benefits were estimated as the reduction in the cost impacts of failure. These were calculated by estimating the current probability of failure and the reduced probability of failure with the safety management practices implemented. These were multiplied by the costs of failure derived from the RFM work.

Proportionality and reservoir reform

Table 2. Application of Safety Management Practices to Operating Reservoirs

Hazard Class	Reservoir Safety Case	Risk Assessment ¹	RSMP	Surveillance	Monitoring	Maintenance	Operating Procedures	Record Keeping	Emergency Planning	Panel Engineer Supervision	Periodic Safety Review	Incident Reporting	QCE Oversight
1	Y	Y	Y	Y ^c	Y ^c	Y	Y	Y	Y	Y	Y ^d	Y	-
2	Y	Y	Y	Y ^b	Y ^b	Y	Y	Y	Y	Y	Y ^d	Y	-
3	Y ^a	Y	Y ^a	Y ^b	Y ^b	Y	N*	Y	Y	Y	SE	Y	-
4	N*	N*	N*	N*	N*	N*	N*	N*	N*	N*	N*	Y	-

a. A more basic version than the standard would be required
b. Basic monitoring or surveillance informed by a Tier 1 or Tier 0 risk assessment
c. Enhanced monitoring/ surveillance informed by a Tier 2 risk assessment
d. At intervals of no longer than 10 years
SE Only when called by SE
N* Not mandatory but considered best practice

The overall Benefit Cost Ratio for applying the recommended approach to all reservoirs in England and Wales, including lowering the threshold to 10,000m³, was estimated to be 9.3, i.e. the benefits of the proposed changes were 9.3 times the estimated costs.

PANEL ENGINEER'S COMMITTEE

Formation

The Panel Engineer's Committee (PEC) was formed in 2022 by Richard Coackley, the then Chair of Reservoirs Committee, under the umbrella of the Institution of Civil Engineers (ICE). His decision arose from concerns within Panel Engineer community about lack of voice in decisions and debates regarding reservoir safety.

The purpose of the PEC was to provide:

- a focus to the voice of engineers appointed to the various panels formed under the Reservoirs Act,
- proactive advice on how best to protect the public from the risk of flooding due to dam failure,

¹ Risk assessment requirement would relate to Hazard Class with a minimum of Tier 2 expected for Class 1, Tier 1 for Class 2 and Tier 0 for Class 3.

- feedback, comment and advice on the implementation in England of the recommendations of the Independent Reservoir Safety Review Report.

Involvement with Reservoir Reform Programme

The PEC has been involved with four of the workstreams of the Reservoir Reform Programme. This paper is limited to the engagement with the hazard classification workstream.

Involvement with Hazard Classification Activity

Three members of the PEC were on the High-Level Engagement Group (HLEG) set up by the project team. The PEC members participated in the project teams meetings with the HLEG. Written feedback to the research contractor included the important recommendation that the benefit-cost ratio of any proposed changes should be separated out by hazard class, rather than an overall average for all dams.

In addition, the PEC facilitated two workshops on issues relevant to the research. One investigated opinion on risk assessment processes and methodologies and the second covered design reviews and how they could be included in the safety management practices.

PEC CONCERNS

The PEC was invited to attend the presentation on the outcomes from the research project on 14th February 2025. Following that the ICE secretariat wrote to Defra on behalf of PEC expressing concerns about proportionality, causality and the inclusion of small raised reservoirs.

Proportionality

Given the high ratios presented, the PEC had concerns regarding the calculations behind the reported Benefit Cost Ratios. At the time of the February workshop that information was not available.

The PEC was also concerned that, from observation, the regulatory burden on Class 3 reservoirs appeared disproportionate: 11 of 12 safety management practices applied to all hazard classes while the consequence was two orders of magnitude lower.

Causality

No information was presented to show the causal path by which the proposed changes in safety management practices would reduce the probability of failure.

Proportionality and reservoir reform

Small Raised Reservoirs

The PEC were surprised that the inclusion of Small Raised Reservoirs had a positive benefit ratio. Research done for Defra by Mott MacDonald in 2021 suggested that including Small Raised Reservoirs under current legislative requirements had a negative balance of value to reduction of risk to life whereas now, with increased requirements and, presumably, costs, the benefit was significantly positive.

Minimum dam height for regulation

The PEC letter of 2025 to Defra suggested that dam height should be defined as above the riverbank, not the bed of the river. The need to use representative height to define the threshold for regulation is one of the findings in Defra research report FD2701 (2020b) that *"the current 25,000 m³ volume threshold should be retained, but that, if there is a wish to regulate reservoirs smaller than 25,000 m³, these reservoirs should be selected on the basis of height."* (Defra 2020b)

Defra Response

Defra responded noting the concerns raised and noting that the information presented in the research reports was research and development proposals and not Government policy.

PUBLICATION OF RESEARCH REPORTS

The research reports were published in mid-July 2025, allowing the assumptions and calculations behind the material presented in February to be reviewed. The information reviewed confirmed the concerns expressed by the PEC.

Proportionality

Benefits

As noted earlier, the benefits derived from the proposed new hazard classification and safety management practices were calculated as the reduction in the consequences of failure: essentially the reduction in probability of failure multiplied by the cost of the consequences of failure, both lives lost and property damage from the RFM work.

Given this methodology, the robustness of the dam failure probabilities is crucial to the benefits calculations.

Table 3 shows the annual dam failure probabilities used for four different dam categories recommended in Floods and Reservoir Safety. The numbers assumed for the Not High-Risk reservoirs and for the unregulated small raised reservoirs are also included. Numbers are given for two scenarios:

optimistic and pessimistic. Optimistic probability estimates assume proactive management and favourable conditions, while pessimistic estimates reflect potential delays or adverse circumstances.

The approximate number of reservoirs in each category are also included in the table.

Table 3. Annual Probabilities of Failure for existing regulatory system (Source: Table 5.1, FD2739)

Category	Population	Dam Failure Probability	
		Optimistic	Pessimistic
A	300	7.215×10^{-5}	1.815×10^{-4}
B	600	8.198×10^{-5}	2.060×10^{-4}
C	700	1.017×10^{-4}	2.550×10^{-4}
D	200	1.017×10^{-4}	2.550×10^{-4}
Not High Risk	300	2.000×10^{-4}	5.000×10^{-4}
SRR	3000	2.000×10^{-3}	5.000×10^{-3}

To assess the realism of these percentages, they were multiplied by the number of reservoirs within each category to give the annual probability of the failure of a Category A, B, C or D reservoir. Multiplying that number by 90 years gives the number of reservoirs of each category that the failure probability predicts should have failed since the introduction of the 1930 Act. The output of these calculations is given in Table 4.

Table 4. Implied number of dam failures

Category	Annual No of Failures		Failures over 90 years	
	Optimistic	Pessimistic	Optimistic	Pessimistic
A	0.02	0.05	2	5
B	0.05	0.12	4	11
C	0.07	0.18	6	16
D	0.02	0.05	2	5
Not High Risk	0.06	0.15	5	14
SRR	6	15	540	1350

These numbers appear unrealistically high compared to the number of failures of regulated reservoirs that have occurred over the past 90 years.

Commentary within the report suggests that the percentages relate to incidents rather than failures. If that is the intent, then the cost of failure derived from the RFM work should not be used in the benefit calculation. The RFM output relates to the consequence of dam breach, not the costs of an incident at a reservoir. The consequences of breach include fatalities where they would occur and total destruction of both residential and non-

Proportionality and reservoir reform

residential property. There have been no fatalities due to a dam incident since the introduction of the 1930 Act.

The estimation of the benefits of the proposed hazard classification and safety management system in the Regulatory Impact Assessment appear to be flawed and, consequently, the benefits are potentially significantly overstated.

Costs

The information also included estimates of the costs of the proposed safety management practices across the hazard classes.

The proposed hazard classification system outlined earlier has an order of magnitude difference in consequence between the hazard classes. This means that the consequence of failure of a reservoir in class 3 is 100 times less than a reservoir in class 1. Therefore, to be proportionate the regulatory burden on a Class 3 reservoir should be 1% of that on a Class 1 reservoir.

Table 7.2 in the Regulatory Impact Assessment Evidence Report gives the 50-year present values of the estimated costs and benefits of the proposed hazard classification and safety management practice regime. Dividing those cost estimates by the number of reservoirs assumed to be in each class gives an estimate of the Undertaker Cost per reservoir.

The reservoir numbers used were for both currently regulated reservoirs and the number of small raised reservoirs. The nearly 3000 small raised reservoirs identified in England have been allocated between hazard classes based on the distribution of the small raised reservoirs in Wales.

Table 5. Regulatory cost for Class 1 and Class 3 reservoirs for England

Category	Class 1	Class 3	Source
50-year PV Undertakers Costs	£328M	£630M	Table 7.2 in FD2739
No of reservoirs	292	1149	Section 2.3 of FD2739
PV Cost/Reservoir	£1.12M	£0.54M	Costs divided by number of reservoirs
Ratio to Class 1 Cost		49%	

The calculation in table 5 suggests that the proposed regulatory requirements on Class 3 reservoirs are grossly disproportionate when compared with the requirements on Class 1 reservoirs.

The disproportionality is not sensitive to the distribution of small raised reservoirs; for example, if all small raised reservoirs are placed in Class 3, the

ratio to the costs for a Class 1 reservoir is 16%; i.e. the regulatory costs proposed are 16 times what would be proportionate.

Causality

The probability of failure used for the proposed hazard classification and safety management practices suggest that the introduction of the proposed requirements will reduce the probability of failure of a Category A reservoir that falls into Hazard Class 1 by two thirds (i.e. the probability of failure of a Class 1 reservoir is assumed to be about 33% of that of a Category A reservoir under the current safety regime).

The causal path by which the additional safety management practices required by the proposed approach (which all relate to plans and documents and not physical works) will result in such a significant reduction in the risk to populations and property downstream is not outlined. This is a significant omission. How can the effectiveness of the proposals be assessed if the causal steps by which they are intended to improve safety are not explained?

Small Raised Reservoirs

Although not explicitly stated, the proposals indicate that the benefits of extending regulation to small raised reservoirs outweighed costs. This is the opposite conclusion to that reached in Defra funded research in 2021 which concluded that the estimated costs arising from including small raised reservoirs under current legislation were more than 100 times higher than estimated benefits.

It appears probable that the benefits of regulating small raised reservoirs have been overstated in the current research in line with the systemic overestimation related to the apparent combination of probabilities of incidents with the costs of failure noted earlier.

CONCLUSIONS

The proposed hazard classification and safety management practices included in the research reports published last year appear to be disproportionate both in terms of the benefits claimed for the overall package of measures and within the package between the various hazard classes.

The causal path by which the proposed changes in practices will reduce the risk to populations and property downstream of reservoirs have not been articulated.

Proportionality and reservoir reform

The benefits claimed for bringing small raised reservoirs under regulation are compromised by the issues identified with the methodology and inconsistent with the findings of research given in FD2701 (Defra, 2020).

It is to be hoped that these issues will be addressed before proposals are brought out for consultation.

REFERENCES

Balmforth D. (2021) *Independent Reservoir Safety Review Report*, available at *Reservoir review: part B (2020)* - GOV.UK (www.gov.uk).

Defra (2025a) *FD2739: Task 6 Final Report*, available at:
<https://www.gov.uk/flood-and-coastal-erosion-risk-management-research-reports/developing-a-new-hazard-classification-for-reservoir-safety>

Defra (2025b) *FD2739: Regulatory Impact Assessment Evidence Report*, available at: <https://www.gov.uk/flood-and-coastal-erosion-risk-management-research-reports/developing-a-new-hazard-classification-for-reservoir-safety>

Defra (2020a) *FD2701: Applying a Risk-based Approach and Improving the Evidence Base Related to Small Raised Reservoirs. Summary report*, available at:
https://assets.publishing.service.gov.uk/media/6037784ee90e07055b44b735/14755_FD2701-SummaryReport.pdf

Defra (2020b) *FD2701 Review of the Existing Risk Methodology FD2701 – Objective 3 March 2020*, available at:
https://assets.publishing.service.gov.uk/media/60377d1de90e07055b44b739/14758_FD2701-Objective3.pdf

Institution of Civil Engineers (2016). *Floods and Reservoir Safety*, 4th Edition



The British Dam Society

Sensitivity of reservoir breach flood mapping to erodibility assumptions

C E FRENCH, JBA Consulting

J R BENN, JBA Consulting

P L MORGAN, Natural Resources Wales

G W RICHARDS, Natural Resources Wales

SYNOPSIS

National flood mapping of reservoir failure is used as part of the evidence base to designate large raised reservoirs as high-risk or not high-risk under the Reservoirs Act 1975. For embankment dams, the Environment Agency's Reservoir Flood Mapping (RFM) Specification estimates breach outflow using the empirical equations developed by Xu and Zhang (2009), assuming a high erodibility (HE) dam. This reflects a conservative approach, adopted because detailed information on embankment erodibility is not generally available.

To date, the sensitivity of reservoir flood mapping outputs to alternative embankment erodibility assumptions has not been examined systematically in England and Wales. Natural Resources Wales (NRW), supported by JBA Consulting, has therefore completed reservoir flood mapping for 28 embankment reservoirs across Wales (median capacity 27,000m³) using high, medium and low erodibility scenarios to support the risk designation process. This paper examines how assumed embankment erodibility affects predicted dam failure consequences, including Average Societal Loss of Life (ASLL), Maximum Population at Risk (MPAR) and estimated property damages, and considers the implications for reservoir hazard classification under the proposed new reservoir safety framework.

ASLL is particularly sensitive to assumed embankment erodibility, with lower erodibility scenarios generally producing substantially lower ASLL values than the standard HE scenario. In contrast, MPAR and property damages are less sensitive to erodibility assumptions. Changes in hazard classification are limited within the study sample, although almost half of the reservoirs are already within the lowest hazard class under the standard HE scenario.

INTRODUCTION

A programme of reservoir flood mapping has been undertaken in England and Wales since 2009. The programme was triggered by the Ulley spillway incident in 2007 and the subsequent recommendation by Sir Michael Pitt for

Sensitivity of reservoir breach flood mapping to erodibility assumptions

the Government to produce inundation maps for both large and small reservoirs. Pitt's (2008) recommendation was primarily for emergency planning purposes, although reservoir flood maps also form a key part of the evidence base used in reservoir risk designation.

Reservoir flood mapping in both England and Wales is currently undertaken in accordance with the Environment Agency's Reservoir Flood Mapping (RFM) Specification. The latest edition at the time of writing is Version 1.3, issued in February 2024. For embankment dams, the RFM Specification uses the empirical breach equations developed by Xu and Zhang (2009), assuming high dam erodibility as part of the standard methodology.

The assumption of high erodibility reflects a conservative approach for national scale flood mapping studies, where detailed information on embankment erodibility is not generally available. However, the assumption may be overly conservative for some reservoirs, particularly where embankment composition and construction characteristics may indicate lower erodibility.

To date, the sensitivity of reservoir flood mapping outputs to alternative embankment erodibility assumptions has not been examined systematically for reservoirs in England and Wales. Natural Resources Wales (NRW), supported by JBA Consulting, has therefore undertaken reservoir flood mapping for a number of embankment reservoirs using all three erodibility categories considered by Xu and Zhang: high, medium and low erodibility. This study examines how assumed embankment erodibility influences the predicted consequences of dam failure. Given the role of reservoir flood mapping in risk designation and hazard classification, the study also considers the implications of alternative erodibility assumptions for the proposed new hazard classification system being developed for reservoir safety reform in England and Wales.

BACKGROUND TO BREACH PARAMETERISATION

Within the RFM Specification, assumed dam erodibility influences predicted breach hydrographs through its effect on peak flow and breach formation parameters. These relationships are based on the empirical equations developed by Xu and Zhang (2009) using a dataset of 75 earthfill and rockfill dam failures with sufficient information for regression analysis. Most failures within the dataset were caused by overtopping or seepage erosion/piping. Xu and Zhang assigned each dam a high, medium or low erodibility category based on material composition, compaction conditions, dam geometry, construction time and other relevant construction information. Their

analysis showed that dam erodibility is one of the most important factors influencing dam breaching parameters.

The reservoirs included within the Xu and Zhang dataset are generally larger than many of the regulated reservoirs in Wales. Most dams within the dataset are located in China and the United States, with only one UK reservoir represented (Coedty). Figure 1 compares reservoir volume and dam height for the Xu and Zhang dataset, registered Welsh reservoirs and the reservoirs included within this study.

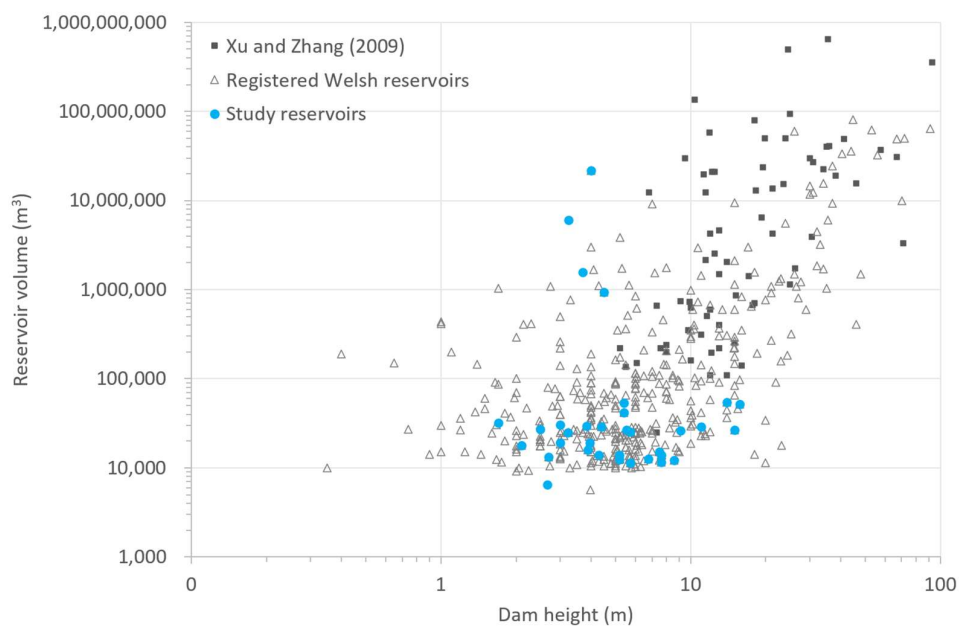


Figure 1. Comparison of reservoir volume and dam height for the Xu and Zhang (2009) dataset, registered Welsh reservoirs and study reservoirs

The Welsh reservoir stock includes many reservoirs smaller than those represented within the Xu and Zhang dataset, with the study sample mostly drawn from the smaller regulated reservoirs in Wales identified through recent reservoir registrations. This introduces an additional source of uncertainty beyond the conservatism associated with the high erodibility assumption alone, as the applicability of the Xu and Zhang breach equations to smaller UK reservoirs has not been examined in detail. However, consideration of the wider applicability of the equations is beyond the scope of this paper.

Sensitivity of reservoir breach flood mapping to erodibility assumptions

The key breach parameters within the RFM Specification influenced by erodibility are peak flow (Q_{xz}) and breach height (H_b). The RFM Specification uses equation 20 from Xu and Zhang to define peak flow, Q_{xz} ($m^3 s^{-1}$):

$$\frac{Q_{xz}}{\sqrt{g}V_w^{5/3}} = 0.133 \left(\frac{V_w^{1/3}}{H_w} \right)^{-1.276} e^{(b_4+b_5)}$$

where:

g = acceleration due to gravity ($m s^{-2}$)

V_w = reservoir volume at time of dam failure (m^3)

H_w = water depth in reservoir above breach invert (m)

b_4 = failure mode coefficient

b_5 = dam erodibility coefficient.

Equation 11 from Xu and Zhang is used to define breach height, H_b (m):

$$\frac{H_b}{H_d} = b_5 - 0.025 \left(\frac{H_d}{H_r} \right)$$

where:

H_d = maximum dam height (m)

H_r = reference dam height (15 m)

Breach height is then used in the time to failure, T_f , calculation set out in the RFM Specification, based on Froehlich (2008):

$$T_f \text{ (seconds)} = 63.36 \sqrt{\frac{V_w}{gH_b^2}}$$

The variability in coefficient b_5 between the three erodibility classes, as shown in Table 1, results in substantial differences in predicted breach hydrographs for equivalent reservoir characteristics. Different b_5 coefficients apply to the peak flow and breach height equations. Based on the coefficients presented in Table 1, the predicted peak flow for a high erodibility dam is approximately 1.5 times that of an equivalent medium erodibility dam and approximately 3.8 times that of an equivalent low erodibility dam.

Table 1. Xu and Zhang (2009) dam erodibility coefficients

Dam erodibility	Peak flow coefficient, b_s	Breach height coefficient, b_s
High (HE)	-0.089	1.072
Medium (ME)	-0.498	0.986
Low (LE)	-1.433	0.858

METHODOLOGY

Reservoir flood mapping has been undertaken for 28 embankment reservoirs as part of a wider rolling programme of RFM for NRW. The reservoirs are distributed across Wales, from Flintshire in the north to the Vale of Glamorgan in the south. The reservoirs range in size from 6,500m³ to over 21.8 million m³ and have dam heights ranging from 1.7m to 16m. The median capacity and dam height are 27,000m³ and 4m, respectively.

Flood routing has been undertaken using JFlow®, a 2D hydraulic model which solves the full Shallow Water Equations using GPU technology. Alternative erodibility scenarios have been implemented through adjustment of breach hydrograph parameters. Peak flow, breach height and time to failure have been recalculated for each scenario using the appropriate coefficients presented in Table 1. For each reservoir, breach simulations have been undertaken for high, medium and low erodibility (HE, ME and LE) scenarios under both dry day and incremental wet day conditions (wet day dam break with fluvial flooding removed). The HE scenario represents the current standard approach within the RFM Specification, with ME and LE scenarios tested as sensitivity cases. Hydraulic modelling, mapping and consequence metrics have been prepared in accordance with the requirements of the RFM Specification.

This paper focuses on three key consequence metrics produced as part of the RFM process: Average Societal Loss of Life (ASLL), Maximum Population at Risk (MPAR) and direct property damages. ASLL represents the estimated loss of life based on property occupancy and predicted flood hazard severity. MPAR is the estimated population affected by flooding, while property damages provide an indication of the direct flood damages to affected properties. The consequence metrics have been derived using point-based receptors from the 2025 National Receptor Dataset (NRD), as recommended within the RFM Specification.

Sensitivity of reservoir breach flood mapping to erodibility assumptions

RESULTS AND DISCUSSION

Influence of erodibility on consequence metrics

Figure 2 shows example breach hydrographs for high, medium and low erodibility scenarios for one of the study reservoirs. Lower erodibility results in lower peak flows and longer breach formation times. Similar relationships are seen for both dry day and wet day scenarios.

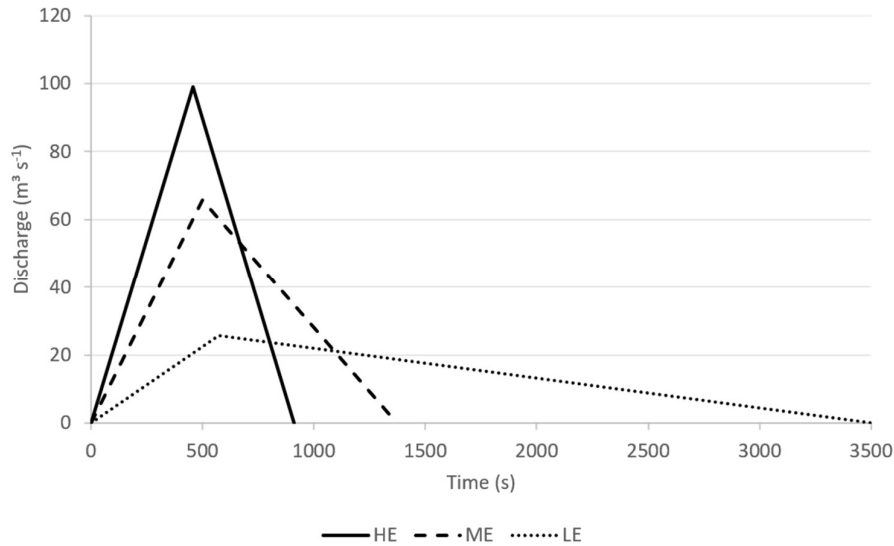


Figure 2. Predicted breach hydrographs for high, medium and low erodibility scenarios under wet day conditions at one of the study reservoirs

ASLL, MPAR and property damage values span a wide range across the 28 study reservoirs, with most reservoirs having relatively low values and only a small number showing much higher values, resulting in strongly skewed distributions (Table 2). This is consistent across the three erodibility scenarios.

The impact of the dam erodibility assumption varies between consequence metrics, with ASLL showing the greatest sensitivity. Both median and maximum ASLL values reduce with lower erodibility assumptions. The reduction is particularly notable for the highest consequence reservoirs, with the highest ASLL across the study reservoirs decreasing from 260 under the HE scenario to 84 under the LE scenario.

Table 2. Summary of ASLL, MPAR and property damages by erodibility scenario

Metric*	Erodibility scenario	Median	Q1**	Q3**	Maximum
ASLL	HE	0.53	0.03	2.65	260
	ME	0.36	0.02	2.21	195
	LE	0.16	0.01	1.19	84
MPAR	HE	77	16	474	9,436
	ME	72	14	475	9,065
	LE	60	14	430	8,161
Property damages	HE	£2.4m	£0.5m	£14.1m	£327m
	ME	£2.3m	£0.3m	£15.1m	£317m
	LE	£1.9m	£0.3m	£12.5m	£284m

* Maximum of dry day and incremental wet day values

** Q1 – Lower quartile (25th percentile); Q3 – Upper quartile (75th percentile)

Ratios relative to the HE scenario provide a clearer comparison between erodibility scenarios, as illustrated by the box plots in Figure 3. ME ratios for ASLL are typically around 0.7 to 0.8, while LE ratios are around 0.2 to 0.4 (Table 3), indicating a substantial reduction in predicted potential life loss under lower erodibility scenarios.

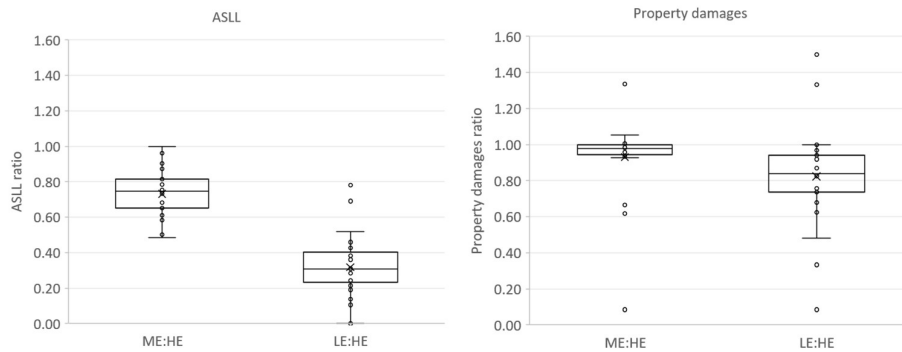


Figure 3. Box plots of ASLL and property damage ratios relative to the HE scenario

Sensitivity of reservoir breach flood mapping to erodibility assumptions

Table 3. Summary of ASLL, MPAR and property damage ratios

Metric	Ratio	Median	Q1	Q3
ASLL	ME:HE	0.75	0.65	0.81
	LE:HE	0.31	0.23	0.40
MPAR	ME:HE	0.99	0.95	1.00
	LE:HE	0.89	0.74	0.97
Property damages	ME:HE	0.98	0.94	1.00
	LE:HE	0.84	0.74	0.94

In contrast, MPAR and property damages show a less pronounced and less consistent response to erodibility. ME ratios are typically around 0.94 to 1.00 for both MPAR and property damages, while LE ratios are generally around 0.74 to 0.97 (Table 3). The greater sensitivity of ASLL to dam erodibility reflects the way the consequence metrics are calculated within the RFM process. ASLL depends directly on $DV_{\max}$ (the product of flood depth and velocity), which generally reduces under lower erodibility scenarios due to the less flashy breach hydrographs and lower peak flows. MPAR depends primarily on the number and occupancy of affected properties, while property damages are determined using relatively broad damage categories based on velocity and DV thresholds. Both metrics are therefore more closely related to overall flood extent than ASLL.

In some cases, property damages are higher under the ME scenario than the HE scenario. Review of the model outputs indicates that this is due to differences in the spatial distribution of flow and flood depths between the scenarios, rather than modelling instability or numerical issues. In these cases, the flashier HE breach hydrograph results in greater depths and more extensive floodplain storage closer to the reservoir than the ME scenario, allowing a marginally greater proportion of flow to be conveyed further downstream under ME conditions. Although these differences are generally small, they can influence whether point-based receptors are included within the consequence totals, even where parts of the same building may be flooded in multiple scenarios. This can result in disproportionate changes in total damages, even where overall flood behaviour is broadly similar. Figure 4 shows a case in point for one of the study reservoirs. Although the overall flood extents are very similar, the ME extent reaches fractionally further into two building footprints than the HE extent, encompassing the NRD points and resulting in the inclusion of two additional properties and therefore higher damages under the ME scenario.

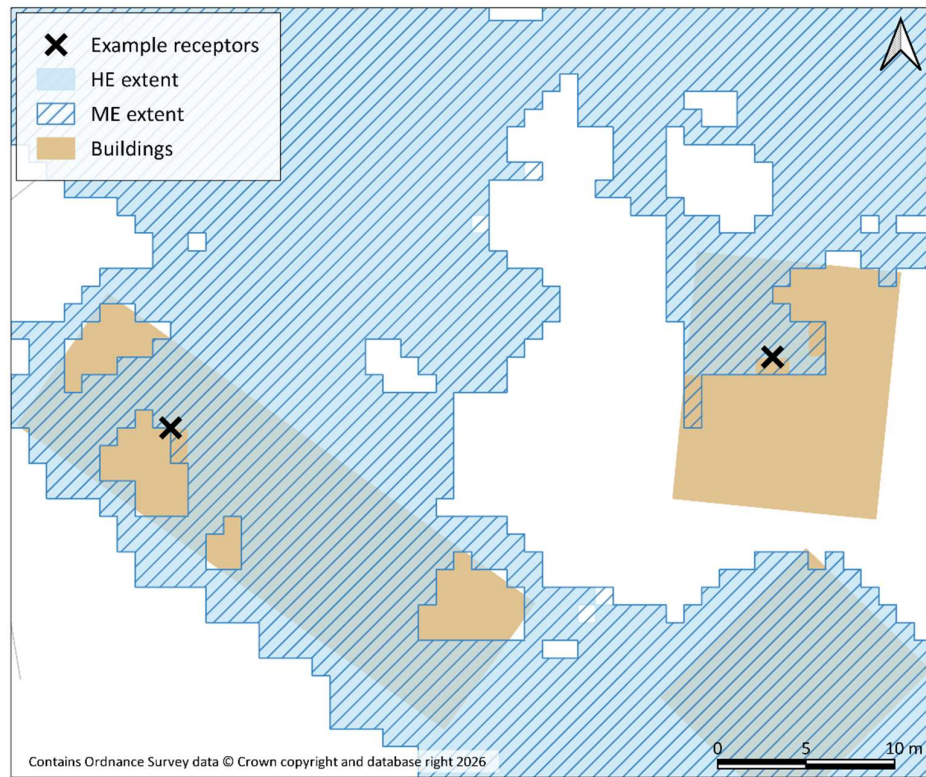


Figure 4. Example of point-based receptor sensitivity, showing receptors included in the ME scenario but not the HE scenario despite similar mapped flood extents

No clear relationship between reservoir size and sensitivity to dam erodibility has been identified. For example, the largest reservoir in the study (>21 million m^3) and the smallest reservoir ($6,500\text{m}^3$) have both produced similar ME:HE and LE:HE ASLL ratios of 0.7-0.8 and 0.3-0.4 respectively. This suggests that factors such as floodplain storage, downstream topography and receptor distribution have a greater influence on sensitivity than reservoir size alone.

Implications for hazard classification

The standard RFM outputs (i.e. the HE scenario) have been reviewed by an Inspecting Engineer. In their opinion, 16 of the 28 reservoirs would be designated 'high-risk' under the amended Reservoirs Act 1975, with the remaining 12 designated 'not high-risk'. The reservoirs have also been classified using the multi-criteria reservoir hazard classification system proposed through the FD2739 R&D project. Using the same HE scenario, 13 reservoirs would fall within Class 4 (the lowest hazard class), with 11 in Class 3, 3 in Class 2 and 1 in Class 1. This demonstrates how the proposed

Sensitivity of reservoir breach flood mapping to erodibility assumptions

framework provides greater differentiation between reservoirs than the current 'high-risk'/'not high-risk' designation system.

The effect of dam erodibility on classification under the proposed system has also been assessed. For most study reservoirs, the proposed hazard classification remains unchanged between erodibility scenarios. However, a small number of reservoirs change class under lower erodibility assumptions, showing that assumed embankment erodibility can influence hazard classification in some cases.

One reservoir changes class between the HE and ME scenarios, while three reservoirs in total change class between the HE and LE scenarios. All move towards lower hazard classifications under lower erodibility assumptions (Table 4).

Table 4. Hazard classification changes for reservoirs affected by alternative erodibility scenarios

Reservoir	HE	ME	LE	Driver metric*
A	1	1	2	ASLL: 260 → 195 → 84
B	3	3	4	ASLL: 1.6 → 1.2 → 0.6
C	2	3	3	ASLL: 11 → 7 → 3

* Metric values are presented in HE, ME and LE order. Bold values indicate the erodibility scenario at which the hazard classification changes.

In each case, reductions in ASLL between erodibility scenarios cause the reservoir to cross one of the ASLL thresholds (100, 10 or 1) within the hazard classification scoring framework. This reduces the associated component score, resulting in a lower overall hazard classification.

CONCLUSIONS

The study demonstrates that the standard HE erodibility assumption used within the RFM process generally produces more conservative consequence estimates than alternative ME and LE scenarios. The influence of erodibility is most pronounced for ASLL, while MPAR and property damages are generally less sensitive. Dam erodibility assumptions have a significant influence on predicted ASLL values and therefore have the potential to affect the position of reservoirs relative to societal risk acceptance criteria bands on F-N plots for a given dam failure probability.

The influence of dam erodibility on the proposed hazard classification system is limited within the study sample, affecting only 11% of the reservoirs. However, almost half of the study reservoirs fall within Class 4 under the HE scenario, suggesting that the influence of erodibility

assumptions on hazard classification may be greater in datasets containing a larger proportion of higher consequence reservoirs.

Given the influence of embankment erodibility on consequence metrics, particularly ASLL, consideration should be given to building flexibility into the RFM Specification to allow alternative erodibility scenarios to be assessed where deemed appropriate by the Inspecting Engineer. In tandem, it is recommended that Inspecting Engineers comment on embankment erodibility as part of their Section 10 reports. Developing a framework for assessing and reporting erodibility would help promote consistency between Inspecting Engineers. The factors considered by Xu and Zhang (2009), including embankment material, compaction conditions, dam geometry, construction period and slope protection, could provide a useful basis for such a framework.

Future updates to the RFM Specification should also consider whether the continued use of point-based property data remains appropriate, or whether building footprints could provide more representative consequence estimates.

NRW is extending the modelling of alternative erodibility scenarios to around 40 further reservoirs in 2026/27 and a similar number in 2027/28. These include a number of larger reservoirs being remapped to the latest RFM Specification, which will provide additional variety within the dataset and help further develop understanding of the influence of erodibility assumptions on the predicted consequences of dam failure.

REFERENCES

- Environment Agency (2024) *Reservoir Flood Mapping Specification. Report - ENVFCPMM00606B00C. Version 1.3 February 2024*. Environment Agency, Bristol, UK
- Froehlich D C (2008) Embankment dam breach parameters and their uncertainties. *ASCE Journal of Hydraulic Engineering*. **134(12)**, pp. 1708-1721
- Pitt M (2008) *The Pitt Review: Lessons learned from the 2007 floods*, Cabinet Office, London.
- Xu Y and Zhang L (2009). Breaching parameters for earth and rockfill dams. *Journal of Geotechnical and Geoenvironmental Engineering*, **135(12)**, pp. 1957-1970



The British Dam Society

Changing Legislation; Practical implications, processes and policies for the retrospective registration of 10,000m³ – 25,000m³ reservoirs.

STEVE MORRIS, Natural Resources Wales
ALUN DAVIES, Natural Resources Wales
JO PARKINSON, Natural Resources Wales
MATT COOMBS, Binnies UK Limited
PAUL RISDON, Binnies UK Limited

SYNOPSIS

In light of the potential changes to the Reservoirs Management through the Reservoir Safety Reform programme and the implementation of small raised reservoirs with a volume between 10,000m³ to 25,000m³, this paper outlines the experience in Wales since 2016, of the practical implications for a reservoir owner to consider when retrospectively registering reservoirs of all types (mining, flood alleviation, National Nature Reserve and water supply). How to prepare for changing legislation and subsequently ensuring they meet safety requirements going forward. In addition, it considers other non-statutory reservoirs and associated policies for management.

This includes a step-by-step run through of the statutory processes included with registration, initial S10 or S8 inspection (depending upon date of construction) and final certificate, potential requirements post initial inspection including likely MIOS or MIOMs that will be specified, associated capital and maintenance costs that can be expected over the initial 10 years following registration, operational considerations, portfolio management (where applicable), problems / issues encountered (and how to overcome them) together with overall timescales that an owner should expect to bring their reservoir in line with the standards expected under the Act.

LEGISLATION

The Reservoirs Act 1975 (RA75) is the law which lays down the minimum requirements for the safe operation and management of large raised reservoirs in Wales. The Flood and Water Management Act 2010 was implemented following the recommendations made by Sir Michael Pitt following extensive flooding in 2007. Schedule 4 of the 2010 Act allowed changes to the RA75 and provided opportunity for Ministers to make regulations to support it.

Changing Legislation; Practical implications, processes and policies for the retrospective registration of 10,000m³ – 25,000m³ reservoirs.

In Wales, the Minister for Natural Resources approved amendments to the RA75 and its regulations, which came into force on 1 April 2016. The steps taken by Welsh Government to amend the regulations are a reaffirmation that reservoirs hold a public safety risk which justifies primary legislation.

NRW is the relevant enforcement authority for the RA75 in Wales, as well as the second largest reservoir Undertaker in Wales. NRW reservoirs serve a variety of purposes including flood alleviation, conservation, water supply and amenity. Following its formation in 2013 a project was undertaken to identify the number of potential reservoirs in NRW's ownership.

Following a GIS based desktop study, potential sites were visited with surveys undertaken to determine whether the waterbody had a raised volume in excess of 10,000m³. This project identified 74 sites which required further assessment by an All Reservoirs Panel Engineer (ARPE) who visited each site to confirm that the structure was a raised reservoir, to assess if the raised volume is likely to be above 10,000m³ and to provide a provisional risk category. It was also an opportunity to identify any obvious works that would be required to bring the structures into compliance with the Act if deemed high-risk and over 10,000m³.



Figure 1. Typical NRW Reservoir Asset, in 2014.

NON-STATUTORY RESERVOIRS AND LEGAL POSITION

By 2015 NRW had 47 registered large raised reservoirs, with a further 27 reservoirs with a capacity below 10,000m³, and in some cases only marginally below this volume. At the same time, the risk designation process (carried out by the NRW Reservoir Regulation team) was underway, resulting in several sites being designated as not high risk. As a competent

Undertaker, NRW decided that it was imperative to maintain its entire reservoir stock, registered or otherwise, in a consistent and transparent manner.

Although there is no legal requirement under the RA75 to undertake surveillance of reservoirs outside of the Act, there remains a Duty of Care upon NRW to maintain these assets and manage the risk posed to others in addition to their duties under the Health and Safety at Work Act.

Additionally, taking into consideration NRW's role and responsibilities as the Welsh Reservoirs Enforcement Authority, NRW is rightfully expected to set an exemplar precedence for other 3rd party reservoir owners or asset managers. Consideration was also given to the wider implication of asset failure beyond solely the safeguarding and protection of human life, including environmental risks and potential impacts, as well as the socioeconomic and reputational impacts of asset failure.

Discussions were held with other reservoir Undertakers such as the Canal and Rivers Trust to establish what procedures they have in place regarding their own reservoirs which fall outside of the relevant reservoir act.

Within NRW the in-house Reservoir Supervising Engineers (SEs), Reservoir Compliance Manager (RCM), NRW's primary ARPE, key reservoir support staff and the Enforcement team representatives were consulted, and it was decided to group NRW assets into four different types with agreed maintenance and surveillance regimes dependant on differing risk profiles:

Table 1. NRW Reservoir Portfolio Group

Group	Type	Description
1	>10,000m ³ HR	Above 10,000m ³ capacity and designated High Risk by Enforcement Authority
2	<10,000m ³ Elevated Risk	Below 10,000m ³ capacity and deemed by NRW Operations to pose an elevated risk in event of failure
3	>10,000m ³ NHR	Above 10,000m ³ capacity and designated NOT-High Risk by Enforcement Authority
4	<10,000m ³ No Elevated Risk	Below 10,000m ³ capacity and deemed by NRW Operations NOT to pose an elevated risk in event of failure

RESERVOIRS ACT REQUIREMENTS

Following identification of reservoirs within the NRW portfolio that could come within the remit of the >10,000m³ requirement, the issue then arises as to how these reservoirs would be registered or notified to the

Changing Legislation; Practical implications, processes and policies for the retrospective registration of 10,000m³ – 25,000m³ reservoirs.

Enforcement Authority, retrospectively. This was undertaken via Schedule 1 form, giving the name of the reservoir and other relevant details such as probable capacity and national grid reference.

It is important to note that some of these details were provisional as the amount of information available on some reservoirs, which were extremely overgrown, was next to nothing (no historical records other than that which could be obtained from historic OS maps from the National Library of Scotland going back to ~1850s and the publication of the Welsh Tithe Maps in the 1830s).

Perhaps the most important item to note is that Schedule 1 is completed after an initial inspection. Within this form for the older reservoirs is the section under 'no certificate: No Final Certificate being issued under the Act of 1930 because the construction was complete before the commencement of the Act'. If the reservoir was constructed prior to the 1930s Act, then a Section 10 inspection can be undertaken, although a S26 annex is required. If the reservoir is constructed after the 1930s Act a Section 8 inspection is required and a final certificate.

It was also agreed best practice to register any breached or failed reservoirs in order that they would be discontinued properly and a subsequent certificate issued.

ADDITIONAL FACTORS

Although there is no legal requirement under the Reservoir Act 1975 (as amended) to undertake specific reservoir risk reviews with regards to individual or portfolios, a number of guidance documents have been released associated with the subjects with a greater number of Undertakers completing various reviews.

The principal guidance documents are as follows;

- Interim Guide to Quantitative Risk Assessment for UK Reservoirs 2004
- Risk Assessment for Reservoir Safety management 2013
- Scoping the risk assessment process for small reservoirs 2010

As NRW also acts as the Enforcement Authority with regards to reservoirs within Wales, they should set an exemplar precedence for other owners or asset managers.

The risk assessment process can have a number of benefits:

- Aligns with good practice regarding periodic risk reviews and the safe management of assets

- Provides an understanding of the risks associated with specific dams
- Facilitates the execution of the Undertaker's duties as detailed within the applicable Reservoir Act
- Identifies areas that may require additional investigations or thoughts
- Facilitate the identification of business risks (non-safety issues) and informs business/investment planning and decisions
- Assists with decision making regarding benefits and priorities
- Proactive approach towards surveillance

The analysis identified above and more commonly undertaken by Undertakers involves specific reservoirs. Often reservoir assessments are an involved process requiring substantial inputs from all parties in terms of time, data and cost.

In consultation with NRW's ARPEs and in-house SEs it was decided to develop a high-level assessment tool to help identify those reservoirs which are likely to pose the greatest risks to NRW, which could be completed relatively easy without the need for substantial data and time inputs.

The review considered all reservoir assets within the NRW portfolio irrespective of size, designation, statutory requirements, outstanding MIOS or function as examples. The driving rationale is that NRW is a public facing organisation that operates to serve and protect its communities and visitors who have the right to explore its assets in safe and managed environments.

It was decided that the risk considerations should not solely focus on the danger to life but also that of environmental, socioeconomic, and cultural, taking inspiration from the SEPA's risk designation factors and reputational consequences of failure. This approach aligns with NRW's overall aspirations, particularly as many of the reservoirs were deemed not high risk or fell below the threshold but still posed for example an environmental pollution risk due to heavy metal contaminations. Should an uncontrolled escape of water occur, the loss of a SSSI or damage to a World Heritage Site was deemed unacceptable.

Changing Legislation; Practical implications, processes and policies for the retrospective registration of 10,000m³ – 25,000m³ reservoirs.

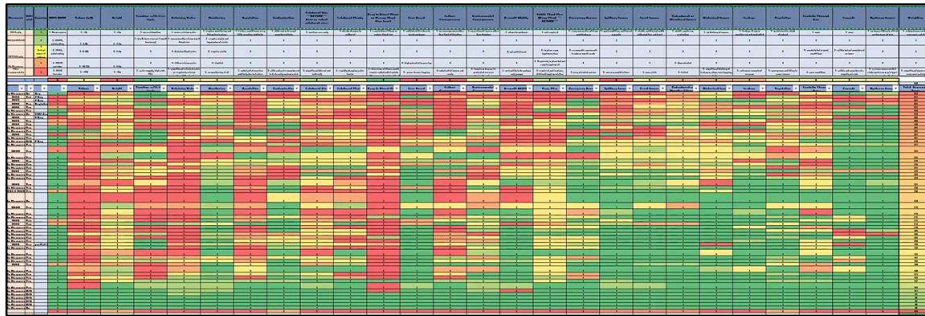


Figure 1. NRW Portfolio Risk Review

The tool reviewed twenty-five attributes including; contamination risk, ease at which flows can be managed, emergency drawdown ability, within a cascade. Each attribute was scored out of five, a score of one was given for the “safest” dams e.g. Cascades 1 – no part of cascade, to 5 – within a cascade and considered a significant risk e.g. an upstream dam had significant issues which could cause issues for the dam in assessment. Additionally, each of the 25 attributes were assigned a weighting score out of 5 e.g. the reservoirs function had a lower weighting than contamination risk. As works to each dam in the portfolio are completed their respective risk rating is reassessed.

Generally, it was considered that the results did broadly align with expectations. Perceived anomalies were investigated and through discussions it was deemed that the resultant higher or lower risk ranking was actually more appropriate than one may first consider when looked at in a more pragmatic controlled manner. It was recommended that;

1. The portfolio risk review is revisited annually, exclusive of the Reservoir Act obligations.
2. Assets deemed higher risk are reviewed in more detail potentially, utilising the RARS, or Small Reservoirs approach identified above.
3. The information is used to inform investment planning for capital works.
4. The information is used to inform investment planning for maintenance works.

The risk assessment tool was used to help inform the surveillance regimes as discussed above, in particularly with respect to operational risks of failure. The surveillance regimes once again took inspiration from SEPA classifications where assets were visited only by SEs or trained reservoir keepers who would elevate concerns should they arise, as SEs typically do with ARPEs. The differing classifications made use of the “slim line” S12

reports or NRW's formal Reservoir Keeper records used as the annual report rather than the routine surveillance record as they do for the High-Risk Statutory Reservoirs.

The risk assessment tool provided additional business case support to interventions at reservoirs where the capacity was below the threshold but yet due to their physical characteristics within their locality were deemed to hold risks which NRW could not accommodate.

Generally, it was found that those above the threshold deemed Not High Risk which NRW considered exhibited elevated risk were due to the pollution risk they held or the loss of habitat which would result from a dam failure.

ESCAPABLE VOLUME

Many reservoirs contained high levels of silt and therefore the issue of escapable volume was significant. Guidance is given on page 27 of the Guide to the Reservoirs Act on how the volume should be defined and suggests that it should be based on spillway crest level and should include the volume of any unconsolidated silt. Following various discussion within NRW and advice from ARPEs it was concluded that there would be some uncertainty within the silt volume estimates from surveys with likely errors +/- multiple percentages (which proved to be the case on several sites). This was therefore considered when determining whether a reservoir should be registered. If the calculated volume for the reservoir site was borderline or just below 10,000m³ the decision was made to register it.

REQUIREMENTS, TIMELINES AND COSTS

In summary the reservoir identification, initial inspection, registration and subsequent addressing of MIOS identified and getting the reservoir up to an acceptable level of operation, maintenance, and risk liability took at least 5+ years per site. This generally included surveying, modelling, maintenance and temporary works. The next Section 10 inspection was undertaken with the results of modelling and survey available thus enabling the development of final capital schemes to address reservoir safety.

The consequences of the change in legislation and review of reservoir stock within NRW resulted in the number of reservoirs registered under the Act increasing from 11 to 41. Due to the costs involved and a large portfolio of reservoirs this resulted in a rolling programme of reservoir capital projects which is still ongoing in 2026, 14 years after the first identification of the reservoirs. This has involved a rolling commitment in the order of £2.5m a year and an opex (maintenance) budget of ~£0.5m a year.

Changing Legislation; Practical implications, processes and policies for the retrospective registration of 10,000m³ – 25,000m³ reservoirs.

Funding is obviously a major issue, with the fact that the reservoirs were largely registered over a 1-2yr period leading to multiple S10/S8 inspections and MIOS with concurrent deadlines, e.g. 2017 there were 54 MIOS deadlines across the portfolio. It also involved a multiple S10 programme due to some S10 next inspection periods being set to three or five years due to the issues raised within the reports. Funding was sought on a yearly basis but with a 5yr forward plan / programme prioritised relative to risk.

PORTFOLIO MANAGEMENT

As a competent undertaker NRW made the decision to manage its reservoirs by applying the RA75 and the risk management approach on a sliding scale for all raised bodies of water in its portfolio. By using this method NRW can clearly demonstrate a consistent and fair approach to management, maintenance and operation of all its reservoirs.

EMERGENCY RESPONSE

The registration of NRW reservoirs in 2015 coincided with a particularly wet and unsettled period, including several prominent named storm events. Many of the reservoirs were in a poor state of repair, having not been maintained to the necessary standards (if at all) since they were abandoned in the late 19th century, with close examination and inspection of the embankment and spillway features proving challenging due to the presence of mature vegetation growth. The spillways were in particularly poor condition in terms of integrity, as well as having inadequate capacity and often in many cases minor breaches. In addition, the lack of necessary embankment freeboard due to historical settlement and any design consideration at the time of construction was of concern.

Several reservoir overtopping events were experienced during the storm events in 2015, which necessitated emergency intervention, under the guidance of an ARPE. These interventions included the installation of substantial sandbags structures along embankment crests to temporarily improve and increase the freeboard, as well as installing temporary geotextile on the crest and downstream faces to improve the erosion resistance properties as a short-term measure. The integrity of several spillways were temporarily safeguarded through the installation of plastic sheets and sandbags, which had to be regularly inspected and repaired / replaced over the course of further storm events. Due to the lack of functioning draw off facilities at most of the sites, temporary syphon arrangements were installed, where appropriate.

Prior to undertaking the physical site interventions, new access routes had to be formed at several sites. Whilst the work often entailed reinstating

historical overgrown access tracks, many sites required the formation of completely new routes, often within SSSIs and areas of historical mining, which contained many undocumented and unmarked mine shafts.

The emergency intervention during the stormy period necessitated the mandatory formal reporting of five reservoir incidents to the NRW Reservoir Regulation team. All of the sites which were the subject of intervention and temporary measures during this time have subsequently been rebuilt to current industry standards, including the construction of new spillway structures and embankments with the necessary freeboard, mostly carried out as MIOS from Section 10 reports and subsequently formally certified and signed off by an ARPE. Each of the sites now include a site-specific onsite plan, with real-time reservoir level telemetry and CCTV cameras having been installed at some of the sites.

EXAMPLE RESERVOIRS

Llyn Pendam

Both East and Western dams are and have always been owned by NRW. The use of this reservoir was leased to the Aberystwyth Angling Club who acted as Undertaker since its registration in ~1996. A relic of historic mining activity, the reservoir was unable to maintain an adequate fish stock for angling purposes and therefore the Angling club relinquished its lease to a paddle boarding company with NRW becoming a joint undertaker.



Figure 2. Llyn Pendam Jan. 2023



Figure 3. Llyn Pendam May 2026

Pandora

The original reservoir was retained by a composite earthfill embankment with a peat core, approximately 3.5m high and with a total length of some 50m. The form of construction was near vertical dry-stone wall built from very large blocks of stone on the downstream face and then an embankment behind the wall. The crest was narrow, of the order of 1.5m and was grassed.

Changing Legislation; Practical implications, processes and policies for the retrospective registration of 10,000m³ – 25,000m³ reservoirs.

The upstream sloped at about 2.5 to 1 with a shallow stone wall in the upper part retaining the crest and timber shoring below the waterline.

The safety works completed in 2024 effectively encased the original dam. A downstream stability berm was constructed on bedrock and the dam crest raised to give adequate freeboard. Riprap was placed on the upstream face and the original spillway on the left-hand side of the dam filled in.

At the site of the previous breach, excavated in 1976 for water abstraction, a new spillway and bottom outlet structure has been constructed. The initial S10 gave a capacity ~20,000m³, the bathymetric survey indicated a capacity of 9,234m³ and a surface area of 7,600m² when full to its top water level. Safety works revealed silt levels were deeper than indicated in the bathymetric survey.

A new spillway crest level was installed at breach level rather than original spillway level which would have resulted in a capacity greater than 10,000m³ according to the bathy. NRW decided to retain the reservoir on the register, even though it was just below the 10,000m³ threshold, as the reservoir was in a cascade and there was a known mine shaft further downstream.



Figure 4. Pandora Reservoir November 2014



Figure 5. Pandora Reservoir in 2024 following completed capital works

Lower Ieuan

The original reservoir has a capacity of 37,000m³ with the Enforcement Authority determining it High Risk. It had indications of penstocks and some form of outlet structure, but this was completely overgrown and could not be established without detailed investigation. It was highly likely that works would be required around the spillway/outlet structure and to increase freeboard to comply with the Reservoirs Act. The reservoir was located 450m into a SSSI with no access and highly problematically to undertake any work, or even to regularly visit and undertake surveillance.

Following a review, it was decided to undertake inundation modelling to determine whether the reservoir category was appropriate. This

subsequently proved the reservoir was Category D and following presentation of this information to the QCE/IE they supported the case for the reservoir to be Not High Risk. The evidence was presented to the Enforcement Authority who subsequently agreed, and the reservoir received an official Not High-Risk Designation. However, there are still currently overdue MIOS and work underway to determine the impact on the SSSI heathland from a reservoir failure.



Figure 7. Lower Ieuan Reservoir June 2018

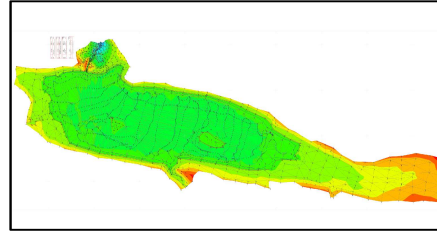


Figure 8. Lower Ieuan Reservoir Bathymetric Survey.

Ongoing Developments

It should be noted that over the last decade NRW has continued to be involved in a number of reservoirs that are either joint owned or have gone back to the Crown and subsequently fallen to the Welsh Government to resolve. Thus, some reservoirs are still at the start of the process of actions post S10/S8 inspections.

CONCLUSION

The experience of NRW from 2014 in preparation for the enactment of the >10,000m³ legislation in 2016, and subsequent decade of S10/S8 inspections, investigations, studies, analysis, maintenance works and capital schemes demonstrated the importance of:

- planning for such change in advance,
- knowing your assets and risks (including outside of legislation),
- being able to implement a quick, simple high level risk review to allow quick decision making,
- assessing all sites and associated risks but being proportional in assessments and actions,
- a high level risk assessment which can assist with decision making regarding benefits and priorities, and provide a proactive approach towards surveillance,

Changing Legislation; Practical implications, processes and policies for the retrospective registration of 10,000m³ – 25,000m³ reservoirs.

- being aware of when the clock starts ticking and the timescales involved (from enactment to first inspection and registration),
- the time constraints and problems of undertaking multiple inspections in short periods, leading to a peak in MIOS and MIOM deadline dated that will provide significant resource and funding problems,
- the options for each which include discontinuance, compliance, enhancement and also challenging the risk designation,
- the timescales actually involved in bringing reservoirs up to a suitable level ~6 years minimum,
- the fact that achieving compliance with the Act can change landscapes through physical intervention at sites (visual impact) and the relationships with local communities,
- the unintended benefits that may arise from works and increased maintenance - an eco-survey has been undertaken for each reservoir, indicating that biodiversity is living in harmony with grass cutting and maintenance regimes, including set aside areas- Water Vole, Medicinal Leaches, Wood White butterfly, Great Crested Newt and hugely enhanced biodiversity area.
- the reputational risks linked to the above.

It is hoped that this paper assists other Undertakers who may find themselves in a similar situation going forward and provide an outline and assurance as to the options and requirements that need to be considered to successfully deal with the challenged posed.

REFERENCES

- Coombs M, Morris S, Davies A, Parkinson J and Isaac P (2024) Multiple Types of Spillway Installation/Refurbishment in Wales (10 years of experience). In *Managing Risks for Dams and Reservoirs – Proceedings of the 21st Biennial Conference of the British Dam Society*. British Dam Society. London, UK
- Morris S, Davies A R and Coombs M (2018) The Multiple uses of NRW reservoirs over the centuries. In *Smart Dams and Reservoirs - Proceedings of the 20th Biennial Conference of the British Dam Society*. ICE Publishing, London. pp 153-165
- Shaw D, Coombs M, Basford A, Morris S and Davies A R (2021). Managing Historical Legacies – Gwydyr Reservoirs. *Dams and Reservoirs* **31(3)** 101-107



The British Dam Society

Large Raised Reservoir 10,000m³ Threshold: Challenges Faced by New Undertakers

L SHAW, Stillwater Associates Ltd
J P HOLLAND, Stillwater Associates Ltd

SYNOPSIS

The 10,000m³ threshold for large raised reservoirs was enacted in Wales in 2016. This paper presents examples of privately owned reservoirs which became large raised reservoirs under the Reservoirs Act 1975 as a result of the threshold reduction.

The owners of these reservoirs in many cases will have been unaware of the possibility that they might face new legislation that would impose a significant and costly burden on them. The owners are unlikely to have the financial means, let alone the requisite knowledge or competence to be able to respond to the legal designation without significant external support. As well as placing an unrealistic burden on the owner, the introduction of these reservoirs has placed an additional burden on the industry which already had a shortage of the necessary resources to support the process.

In some cases, by the very nature of reservoirs being located on watercourses that naturally form property boundaries, the ownership and Undertaking has fallen to more than one landowner. Whilst this might allow the financial burden to be shared, experience shows that this can complicate the process, and it may not be possible to identify all landowners.

The paper will explore the need for potentially providing initial, or even ongoing, financial, technical and legal support to new Undertakers to assist them through the process. Consideration could also be given to having an appropriate competent authority adopt and manage the reservoir, removing the burden from the landowner.

‘NEW’ LARGE RAISED RESERVOIRS, NEW UNDERTAKERS

Under provisions made in the Flood and Water Management Act 2010 the capacity threshold for reservoirs in Wales was lowered to include reservoirs with a stored volume between 10,000m³ and 25,000m³. The process which followed has caused difficulties for many reservoir owners who have unwittingly found themselves to be reservoir Undertakers with legal duties under the Reservoirs Act 1975. Experience suggests that, in general, the outcome of inspections at these ‘new’ reservoirs has been the need for remedial and improvement works, in many cases required as measures in

Large Raised Reservoir 10,000m³ Threshold: Challenges Faced by New Undertakers

the interests of safety. The total financial burden across this group of owners is enormous against a background of limited resources and a lack of crucial legal and technical understanding.

‘NEW’ LARGE RAISED RESERVOIR EXAMPLES

Llyn Ffridd y Bwlch, Blaenau Ffestiniog

The reservoir was originally constructed to support local slate mining. Formed with a masonry dam 4.6m high, 100m long and with a 5m wide crest the reservoir stores 23,000m³ above adjacent natural ground.



Figure 1. Llyn Ffridd y Bwlch Reservoir

Located within an historical mining environment the local fishing club bought the reservoir in 1971 and have access to the bank for fishing. Their ownership specifically excludes the dam, which was retained by the quarry owner. The quarry owner subsequently went into receivership and the ownership of the dam is no longer recorded with Land Registry.

There are MIOS for this Category A reservoir which the fishing club does not have the funds to progress. As things stand there will be no action to address the MIOS until the legal status of the Undertakers is resolved, the process for which is unclear.

Llyn Ffrith Carreg y Bwch, Dolwyddelan

The land which included the reservoir was purchased in the 1960s by a private owner who is now the sole Undertaker. The reservoir impounds 17,112m³ above adjacent natural ground with a masonry dam 103m long

and 6m high. As well as providing a private amenity the owner has historic agreements to supply water to five remotely located residential properties.



Figure 2. The elderly Undertaker inspecting Llyn Ffrith Carreg y Bwch Reservoir dam, precariously perched on the narrow crest

Monitoring and maintenance responsibilities have fallen to the elderly owner. The reservoir is Category C and whilst there are no outstanding MIOS these monitoring and maintenance duties are now beyond the owner's capabilities who currently relies on help from family members. In the longer term the property sale can be expected to be complicated by the presence of the reservoir and the associated liability.

Llywernog Pond, Aberystwyth

Llywernog Pond was discontinued with the S13(2) certificate issued in 1987. Under the 2016 review process the reservoir was confirmed as storing 87,650m³ above adjacent natural ground.

The reservoir is impounded with a 5m high earthfill embankment dam, 80m long, 5m high and has no operable installed drawdown facility.

Large Raised Reservoir 10,000m³ Threshold: Challenges Faced by New Undertakers



Figure 3. View along the dam at Llywernog Pond

The Section 10 report in 2024 concluded that the reservoir is Category B, with recommendations made in the interests of safety for which funding will be required. Although commercial opportunities are being explored there is currently no revenue income from this reservoir.

Llyn Du Bach, Blaenau Ffestiniog

This reservoir, storing 40,710m³ above adjacent natural ground was identified as a large raised reservoir under the 2016 review process. The reservoir had a masonry dam with peat core, 3.3m high and 23m long.



Figure 4. Llyn Du Bach Reservoir, now discontinued

Land Registry indicated three owners who would be Undertakers of which only one was correctly identified initially. This was a hill farmer who bought the land sixty years ago for grazing. The MIOS requirements for, or discontinuance of, this Category A reservoir were well beyond the means of this landowner. A second Undertaker was correctly identified following the Section 10 inspection and reporting. The third was never identified.

The two identified Undertakers both agreed the reservoir served no purpose and therefore should be discontinued as repairs and improvements and ongoing maintenance would be at least as costly. The reservoir was drawn down to make it safe and discontinued over the winter 2025/26.

INITIAL CONTACT IMPACT AND ASSUMPTIONS

Initial letters from the Enforcement Authority must strike a careful balance between being informative, clear, readable, and appropriately formal. Achieving this balance is not easy, and it is evident that significant time and effort has gone into drafting such correspondence, supported by the expertise of the team involved. However, the way this initial letter is received can differ greatly depending on whether the recipient is an individual or a business.

Businesses often have internal expertise or structures and resources in place for legal support to help interpret and respond to the letter. In contrast, an individual may have no clear starting point and limited access to support. The initial letter implicitly assumes that the recipient has the cognitive ability, time, and financial capacity to deal with the issue, as well as access to a computer along with the necessary digital literacy. It also assumes the availability of personal or professional networks to help them understand the requirements of the Reservoirs Act.

While the letter has referenced further resources, these are often limited to the Enforcement Authority's website or government (.gov.uk) pages. It would be beneficial to include signposting to independent support organisations, such as Citizens Advice, or other advocacy services that could assist individuals. Developing a self-help webpage that clearly directs recipients to relevant resources could also be valuable, provided there is sufficient capacity within the third sector to offer meaningful support.

When an individual has unidentified 'others' sharing the reservoir ownership they are obliged to begin a potentially lengthy process involving the Land Registry and local inquiries. Local enquiries within a small community can adversely affect relationships with neighbours. If the Enforcement Authority has been unable to identify all parties, it is unrealistic to expect an individual to achieve better results.

Large Raised Reservoir 10,000m³ Threshold: Challenges Faced by New Undertakers

Whilst the Enforcement Authority has a legal duty to carry out an enforcement role, the timeline for compliance presents further difficulties. From the initial contact through to identifying other owners, registering the reservoir properly and appointing an All Reservoirs Panel Engineer (ARPE) a considerable time may pass. This reduces the time available to meet legal deadlines for registration and inspection and in this context, receiving a formal reminder of legal duties, particularly one that references potential court proceedings, can be distressing for individuals who are already making genuine efforts to comply. Such communication risks being counterproductive and may undermine future cooperation.

A reservoir identified in this context is unlikely to meet current flood risk standards. As a result, it is reasonable to assume that improvement works to achieve compliance may involve substantial costs, which could be potentially life-changing for the individual concerned.

The process places significant demands on individuals in terms of financial resources, personal capacity, and technical understanding. Even chartered civil engineers require years of additional training to operate effectively within this framework. Expecting a member of the public to quickly reach a similar level of understanding is unrealistic.

Finally, it is important to recognise a fundamental distinction between reservoirs that generate revenue and those that do not. For individuals without any income stream associated with the reservoir, the demands of ownership represent not only a financial burden but also a potential strain on mental wellbeing.

DEFICIENCIES, DEFECTS AND NEGLECT

By the very nature of the process to draw in small raised reservoirs (SRRs) under the Reservoirs Act 1975, these reservoirs will tend to be old, historic or even heritage features. Unlike the majority of existing owners of large raised reservoirs, owners of SRRs are highly unlikely to have undertaken much of the necessary maintenance or improvement measures required to keep their reservoirs in a condition that does not present an unacceptably high risk to people and property downstream. In most cases, any money spent on their reservoirs will have been to enhance them as a landscape feature or to improve an amenity value, such as for fishing. Not only might such works not address prevailing reservoir safety shortcomings, in many cases these might add to existing issues or create new problems, such as cutting into the upstream face of a dam to create fishing platforms or modifying the overflow to raise the water level.

It should also be remembered that, in the case of industrial dams such as those intended to support a consistent supply of water for water wheels (Llyn Du Bach), the dam is likely to have been constructed quickly with reliance on regular maintenance or reactive repairs by a local workforce, rather than a robust initial construction based on an adequate design. Once the need for the water supply had gone, the maintenance of the dam will have ended but the dam and associated infrastructure will have been left in place and forgotten about, leaving it to gradually fall into disrepair.

The list of potential issues and deficiencies with SRRs is long and includes:

- poor original construction using locally won variable materials with no core wall and little material compaction (Llywernog Pond).
- Tightly packed dry stone walls to contain a peat infill core (Llyn Du Bach and Llyn y Garnedd), relying on the walls for stability against sliding and overturning (gravity dam) with the peat infill providing the watertight element. Peat is easily eroded by water flow and any defect, even if small, will lead to the peat being washed out from the body of the dam, potentially accelerating the deterioration of the foundation and destabilising the stone walls.
- poorly prepared foundation with deterioration over time from the seepage of water, weakening the interface between the dam material and ground beneath.
- Historic reservoirs may have originally incorporated a draw-off facility (Llyn Du Bach, Llywernog Pond, Llyn Ffrith Carreg y Bwch, Llyn Ffridd y Bwlch) but these outlets may have been blocked off or modified over time. Outlet penstocks, valves or boards of cast iron and timber construction are likely to have been buried in silt or otherwise lost and if found are no longer operable. These will present a weakness within the dam structure leading to leakage and ultimately to internal erosion around the outlet feature or through the lower part of the dam.
- Original outlet facilities, if present and operational, are commonly cast iron valves and pipes passing through the body of the dam or constructed at foundation level. This feature is an inherent weakness with potential for water to pass along the interface between the pipe and the dam fill material resulting in internal erosion. If the outlet valve discharges into a brick lined culvert through the dam at low level the historic brickwork and mortar joints are in constantly damp or submerged conditions which inevitably deteriorate and weaken over time. Open joints or loss of bricks from the lining will allow water under pressure to erode the internal structure of the dam when the outlet is

Large Raised Reservoir 10,000m³ Threshold: Challenges Faced by New Undertakers

operated and may only become evident when the fill material above the culvert shows settlement or sinkholes appear.

- Overflow arrangements at SRRs may take the form of a concrete or masonry lined channel, or as a channel cut through rock around the abutment area (Llyn Du Bach). Often, however, the overflow will be one or more pipes through the upper part of the dam (Llyn y Garnedd, Llywernog Pond). Almost invariably these arrangements will have insufficient overflow capacity, resulting in insufficient freeboard, when assessed against current standards.



Figure 5. Example of inadequate overflow arrangement and freeboard

OWNERSHIP ISSUES

The legal interpretation of an Undertaker for a newly registered reservoir under the Reservoirs Act is primarily determined by land ownership to the reservoir waterline for which the primary source is Land Registry which allows the Enforcement Authority to contact the owners. Any blanks in this register from land sales pre-1990 make finding the landowner very difficult. Without owners stepping forward voluntarily, the only option is legal proceedings, which are costly and time-consuming and offer no guarantee of success; this approach is unlikely to be available to most individuals.

The first example is an old quarry reservoir for water to run cutting machines (Llyn Du Bach). There are three landowners; one a farmer who had registered their land with Land Registry, the second was owned by a working

quarry and the third was a blank on the Land Registry map. Enquiries locally by the quarry and farmer could not conclusively find the third landowner. The quarry and farmer registered the reservoir and initiated the first inspection and appointment of a supervising engineer. The inspection found the dam to be in a poor condition requiring MIOS to either discontinue or undertake substantial repairs.

In the background the quarry legal team was looking at their lease agreement and concluded they had no duties to comply with the Reservoirs Act 1975 and this reverted to the landowner, the Crown Estate. From the time this new Undertaker was made aware of their duties, understanding them and deciding on the best option going forward, many months had passed, time that reduced the MIOS action window. A decision to discontinue by the two undertakers was agreed with the funding for this process left unclear until events on the ground took over and the substantial costs for the discontinuance work were taken on by the Crown Estate.

The second example (Llyn Ffridd y Bwlch) is a reservoir that was bought by a fishing club in the 1970s from a quarry owner. The deed of sale is for the reservoir only and the right to access for fishing, it specifically excludes the dam structure and anything downstream. The quarry went into receivership, with some assets of the quarry bought to operate as a quarry immediately downstream of the reservoir. The Enforcement Authority has not been able to determine the dam owner, leaving the fishing club as the only registered Undertaker. There are costly flood and inundation studies to be undertaken from the first inspection which the club cannot afford to commission. If the required reports lead to dam or spillway improvements, it is unclear if the fishing club have any legal ability to alter something they do not own, even if they could afford it.

The final example (Llyn Crafnant) is a reservoir that was left in trust to a local community council. The Trust does not have sufficient funds to carry out the required MIOS and a meeting on site with the Enforcement Authority to find resolution led to some discussion on the land ownership of one side of the reservoir which is operated by Natural Resource Wales Forestry. It may be that NRW is a joint owner of the reservoir and therefore would have Undertaker duties. Investigations are ongoing to properly identify the Undertakers at this reservoir before MIOS works are progressed.

UNDERSTANDING FLOOD RISK

As with other aspects of reservoir ownership, the level of understanding in relation to flood risk can vary widely between owners. It is apparent that many longstanding reservoir owners do not fully appreciate the magnitude

Large Raised Reservoir 10,000m³ Threshold: Challenges Faced by New Undertakers

of the flood events that must be considered to ensure the safety of reservoirs. Generally speaking, most owners, even those already conversant with their reservoir responsibilities, have little or no appreciation of the potentially life threatening flood hazard to people downstream.

Explaining flood risk to owners can be difficult and time consuming. Explanations are often met with bewilderment or ridicule. A common response will be along the lines of *“we have lived here for thirty years and never seen torrential flows across the overflow”*.

The provision of online information with a detailed presentation of flood risk in relation to reservoirs, in layperson's language, would be a useful addition to the information presented to owners of SRRs, ideally in advance of notices being sent. This information could address the flood impact on reservoirs, and how reservoirs need to be designed to accommodate these floods, as well as the flood risk posed by the reservoir to people and property downstream. It could also usefully reference climate change and how this may increase the risk to the reservoir over time.

Well presented information, available online, could greatly simplify the visit by the Panel Engineer assigned to a first inspection, and would provide a valuable reference to add to the inspection report.

UNDERSTANDING THE RESOURCE REQUIREMENTS

Reservoir ownership brings with it a range of significant time and financial commitments, particularly for individuals who must first develop an understanding of the Reservoirs Act and associated legal duties. This initial learning phase alone can require considerable effort and, in some cases, external support. Beyond this, there are substantial direct costs associated with compliance, including the initial inspection, the implementation of Measures in the Interests of Safety (MIOS) which can require a detailed flood study. There are also ongoing costs for the annual visit and services provided by the Supervising Engineer (SE).

In addition to these defined services, there is an important but less formally recognised requirement for both SEs and All Reservoirs Panel Engineers (ARPEs) to actively inform and educate reservoir owners about their legal responsibilities and the practical steps required to achieve compliance. This often involves extended communication, including additional phone calls, emails, and potentially extra site visits, as well as time spent explaining options and supporting decision-making. Such input goes beyond what is typically included in standard service agreements or initial quotations.

This situation raises two key issues. Firstly, whether there is sufficient capacity within the pool of SEs and ARPEs to provide this level of support to all affected owners. Secondly, on cost allocation—specifically, who is responsible for funding the additional time and professional input required to bring owners up to the necessary level of understanding to help ensure compliance with legislation.

WHAT'S NEXT? HELPING THE INDUSTRY HELP NEW OWNERS

From the experience of working with new owners of recently identified large raised reservoirs it appears that there are a number of aspects of the process that could be developed. With the prospect of the threshold reduction in England, improvements in the process should help new owners and, in doing so, help the industry as a whole.

Initial Contact

The starting point for any newly identified reservoir will be with the Enforcement Authority. Feedback from new owners strongly suggests that a longer lead-in, with earlier and more interactive contact and introductory information would be welcomed. This should include carefully formulated legal and technical advice, clearly outlining the flood risk associated with reservoirs and the need to ensure they are properly and routinely checked and maintained. This proactive rather than reactive process may take years rather than months, but should be seen within the context of the length of time the reservoirs have been in place and the relatively low risk in relation to this timeframe. There may be merit in starting this process well in advance of the threshold reduction being enacted and would allow owners the time needed to take stock, seek advice, plan and budget.

Communication and Information

Improved communication should include a greater level of technical information provided to the owners to help them understand the nature of their reservoir, the potential impacts of floods and the flood risk posed by the reservoir to people and property downstream. Guidance could extend to the various elements of reservoirs and dams, such as the overflow and drawdown arrangements, to help owners understand likely deficiencies, how these can be addressed and the nature and likely costs of remedial or improvement works.

Legal, Land Ownership and Technical Assistance

Since 'new' owners are unlikely to have the necessary legal or technical understanding, there should be an opportunity to provide at least initial

Large Raised Reservoir 10,000m³ Threshold: Challenges Faced by New Undertakers

advice. This input would need to be provided by the Enforcement Authority, working with the reservoir owners, and could, for example, include:

- Initial legal advice specific to the reservoir, including a full explanation of the requirements of the Reservoirs Act 1975.
- Where there is more than one landowner, definitive identification of all those with Undertaker responsibilities, with the legal support necessary to conclude this process.
- More detailed technical support, taking the form of a pre-Section 10 inspection carried out by an ARPE followed up with further explanation and guidance to help the owners understand and plan the monitoring, repairs, maintenance and improvement works that are likely to be required.

Realistic MIOS Timeframes

As the process for new Undertakers unfolds, with the Section 10 Inspection, the Inspecting Engineer (IE) should, with the support of the Enforcement Authority, consider assigning longer timeframes to MIOS. The MIOS process should then adopt a more collaborative approach between the Enforcement Authority and the IE to encourage the Undertaker to act on the MIOS in a timely and properly planned manner. This approach may help avoid the need to threaten enforcement action at any point in the process and help build a more trusting and productive relationship among those involved.

Adoption by a Competent Authority

The change in threshold for large raised reservoirs in Wales has served as a valuable reminder to the industry that many high-risk large raised reservoirs are owned by individuals or groups who do not have the competence or resources to operate and maintain their reservoirs properly to protect those downstream from the prevailing flood risk. In these cases, where the risk to a population downstream is unacceptably high, consideration could be given to having an appropriate competent authority adopt and manage the reservoir, removing the burden from the landowner.

FINAL THOUGHT: LESSONS LEARNED IN WALES BROUGHT TO ENGLAND

Even if the threshold in England ultimately remains at 25,000m³, with the process already underway, at least some owners of SRRs are now being made aware of their responsibility to protect people and property downstream. This education may result in opportunities for improvements in the way SRRs are managed, operated and maintained which in turn should help to reduce the overall risk posed by these reservoirs.



The British Dam Society

Help! I own a Reservoir

RICHARD TERRELL, Binnies
CARL BUGLER, Caerphilly CBC

SYNOPSIS

This paper looks at the journey that the Undertaker took to upskill their staff and become compliant with the Reservoirs Act 1975 and the role the Supervising Engineer can play.

Due to contracts ending and issues with procurement relating to insurance the reservoirs were without a Supervising Engineer (SE) for 12 months. During this time the team who looked after the reservoirs also underwent a change. With advice from Natural Resources Wales (NRW) a SE was appointed and a review of the reservoirs was undertaken. It was identified that there were outstanding Measures in the Interests Of Safety (MIOS) on a reservoir, the Prescribed Form of Record (PFoR) was in the old format, and two reservoirs required inspection after being classified as high-risk reservoir with the change from 25,000m³ to 10,000m³.

Working with the Undertaker a strategy was put into place and agreed with NRW to ensure compliance and improve monitoring. Part of this improvement involved updating the PFoR to the new format, helping to produce flood plans, and providing new monitoring sheets to record water levels, V notch readings and piezometer data, which also graphed the data for analysis. The Council's in-house Engineering Projects Group was also commissioned to help the Client with procurement, delivery and construction works as well as liaising technically with the SE. Finally, on-site training was carried out by the SE to Council staff on monitoring and inspections of the embankments.

HISTORY

Caerphilly County Borough Council (CCBC) is a unitary authority in Wales and covers 277km². It is the Undertaker for five Statutory High Risk reservoirs; three greater than 25,000m³ and two greater than 10,000m³.

Due to contracts ending for SE services, and new personnel in the Countryside department taking over the operation of the reservoirs, it was found that there were some Section 11 breaches under the Reservoirs Act 1975 (the Act), including missing information such as drawings. There were also four MIOS deadlines that were approaching. A Supervising Engineer

Help I own a Reservoir

from Binnies who lives close to the sites was appointed in 2023 and, working with the Undertaker, formulated an action plan to improve compliance.

Historically, the management of reservoir-related works was procured on an annual basis through the Council's Procurement function, requiring recurring tendering and associated administrative processes. This approach was, in part, driven by challenges faced by the Countryside team in achieving and maintaining compliance with the Reservoirs Act, necessitating reliance on support via Procurement. Furthermore, instances of non-compliance had resulted in breaches being formally reported to the Council by the regulator, which acted as an additional driver for seeking a more robust and technically supported management approach. While the procurement-led model was compliant, it introduced inefficiencies and limited continuity in the delivery of specialist services.

In contrast, the Engineering Projects Group (EPG) possesses the necessary in-house skills, experience, and organisational knowledge to undertake these activities. Transitioning towards EPG-led delivery enabled a more streamlined and consistent administrative process, reducing procurement burden while improving coordination, technical oversight, and continuity in reservoir management.

Action plan

First the SE and Undertaker held a meeting to establish the MIOS requirements and these were mostly completed within the time frame. These are not covered any further by this paper.

The focus then turned to Section 11 compliance and improvements in monitoring. Over the course of a few weeks various meetings were held between the SE and staff from CCBC. These focused on training and education of the staff who are responsible for the day to day supervision and the long term management of the reservoirs.

Workshops were held on the requirements of the Act and explanations on the differences of the Section 10 reports and Section 11 Statements and what is legally enforceable. This allowed the Undertaker to learn the "jargon" used. The Undertaker also found benefit from the recommendation to obtain a copy of the Guide to the Reservoirs Act.

Following this, internal discussions were held within CCBC, and the Engineering Projects Group was requested to assist in achieving compliance. This reflected the limited availability of specialist reservoir engineering expertise within the Countryside team, who had responsibility for the reservoirs alongside their existing operational duties. The involvement of the

Engineering Projects Group provided the necessary technical support and oversight to ensure that compliance activities could be carried out in accordance with current standards and best practice.

In parallel, consideration was given and is under development for a more formalised delivery model, whereby EPG would undertake a commissioned role on behalf of the Countryside team, effectively acting as the client's engineering representative for all five reservoirs.

This approach is intended to provide a consistent, risk-based framework for the management of the assets, ensuring ongoing compliance while embedding appropriate technical governance and accountability.

Section 11 compliance

A review of data and records was carried out and a RAG list was produced. The first document to be looked at was updating the Prescribed Form of Record (PFor) into the new format as per SI 2013 No. 1677 (HMG, 2013) as they were in the old "blue book format". The opportunity was taken to also transfer the data into an electronic format of the PFor. This allowed for easy central storage so it does not get lost, ease of transferring the information to the SE and ease of updating with any required updates.

Next on the list was the recording of surveillance data, as the current practice was for a paper sheet to be filled out and put in a folder - so they could be out of date order. This made data analysis difficult and changes in trends could not be spotted. To improve this the SE advised the Undertaker to start using a spreadsheet record that plotted reservoir level, V notch readings and daily rainfall readings from NRW rain gauges. Where relevant, piezometers were also plotted to mAOD along with the water levels. Figure 1 shows the format of the input sheet which highlights the cells to be populated.

A	B	C	D	E	F	G	H	I	J	K
Reservoir Name	XXXXX Reservoir	Rain gauge https://rivers-and-seas.naturalresources.wales/Station/10397								NOTE: FILL OUT GRAY CELLS ONLY
TWL	324.67 mAOD							Top of Tube P1		326.5 Top of 1
		DO NOT TOUCH	RANGERS INPUT	DO NOT TOUCH	RANGERS INPUT	DO NOT TOUCH	RANGERS INPUT	DO NOT TOUCH	DO NOT TOUCH	RANGERS INPUT
Date	Water level (m)	V notch reading (mm) if dry put 0	Water level to mAOD	Piezometer dip 1(m)	Piezometric surface P1 (mAOD)	Piezometer dip 2(m)	Piezometric surface P2 (mAOD)	Rainfall (mm)	Depth of Water flowing over waste weir or overflow (m)	Depth of Water flowing over waste weir or overflow (mm)
01/01/2025	324.67		RN/A		RN/A		320.1	10.2	RN/A	
02/01/2025	324.67	2	324.70	6	320.5	6.4	320.1	9.9	0.03	30
03/01/2025	324.67		324.70		RN/A		RN/A	0.9	0.03	30
04/01/2025	324.67	4	324.70	6	320.5	6.4	320.1	6	0.03	30
05/01/2025	324.67		RN/A		RN/A		RN/A	46	RN/A	
06/01/2025	324.67	4	324.70		RN/A		RN/A	10.9	0.03	30

Figure 1. Example of Excel input table.

The sheets now automatically graph reservoir level against V notch readings and V notch readings against rainfall. Where applicable reservoir levels are plotted against piezometer readings to mAOD.

Help I own a Reservoir

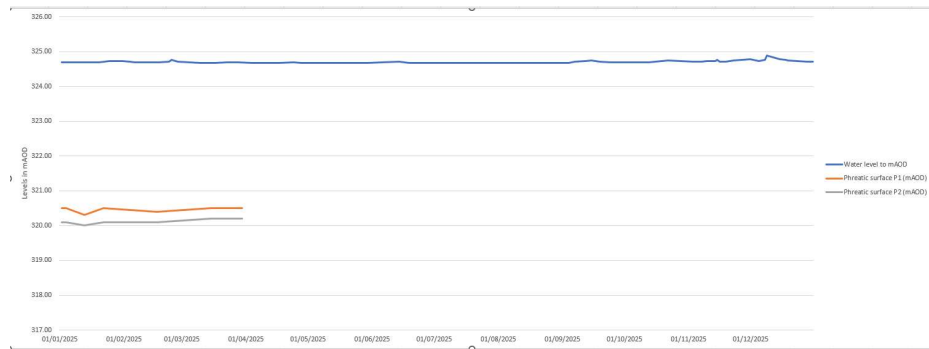


Figure 2. Water level and Phreatic surface plotted to mOAD.

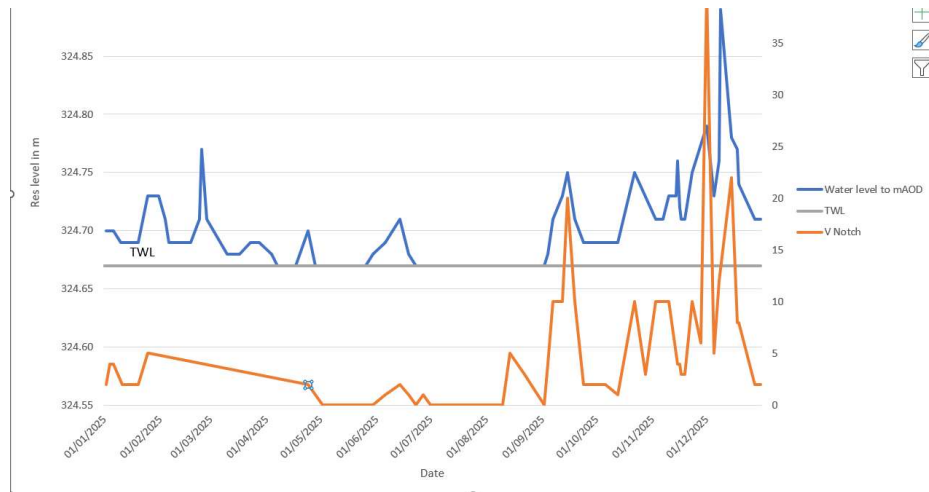


Figure 3. Reservoir level and V notch plots.

Compliance improvement

As work progressed to improve data and understand the behaviour of the dams it became apparent, due to limited data, that there was no understanding of how the dam behaved under different conditions. The last S10 reports had given a minimum frequency for reading V notches and piezometers, some of which were every four months. This was taken as what was needed to be done and no more.

Over the course of the following twelve months the Undertaker's staff were encouraged to take readings on every visit. At first there was resistance, as this was seen as extra to the day job, and the reference was made to the time scales stated in the S10 report. During the workshops it was explained that it was not extra work as they were "walking past the V notch on the day job" and to just take a measurement and note it down. It was also explained why having the information was important and that patterns could be

checked and problems identified quickly, once it was known what range the readings should be.

The Undertaker has now expressed the intention to appoint a dedicated Senior Assistant Engineer to visit the reservoirs and take the levels and readings, and to analyse the information on a weekly basis.

Since the introduction of the sheets, trigger levels have been added to highlight any high readings that may need investigation, such as taking the reading again and highlighting it to the SE.

Onsite plans

Even though not a requirement in Wales it is seen as good practice for all sites to have a flood plan, and producing one was an outstanding MIOS. The Undertaker had no previous experience in producing an on site plan and again they turned to the SE for help and advice. The SE, who will probably be the first point of contact outside the Undertaker's organisation during an emergency, can and should influence the plan to be fit for purpose.

In this case the SE helped produce the plan, working with the Undertaker's staff to confirm site access, valve operations etc. Once the first plan was produced the Undertaker then used this as a template to produce the other plans.

The SE's Role

We are all aware of the SE's role and the requirement of our written statement to "tell the owners and operators and the Enforcement Authority whether statutory records including the Prescribed Form of Record (PFoR) are up to date" and "Is the owner or operator compliant with their duties for monitoring, surveillance and record keeping? Sections 6, 9 and 11 of the act refer. See section 2.4 of the written statement." All as listed on HMG (2022).

As SE's we normally comment and say a "Yes" is compliant or "No" and list what is not there. As the SE is often the only contact some of the Undertakers have with professional Reservoir Engineers, we should also be assisting the Undertakers in how to be compliant. This could be in the form of advising how to take level readings, and when to take the readings.

Helping Undertakers to record that data will lead to making the SE's and IE's analysis easier, as it will be in a usable format.

The Undertakers Role

The Undertaker is responsible for managing and delivering works that interface with the reservoir and associated infrastructure in compliance with relevant standards and statutory requirements. This includes undertaking

Help I own a Reservoir

appropriate risk assessments, coordinating with stakeholders, and implementing measures to protect structural integrity, public safety, and operational continuity, while ensuring accurate records and competent execution of the works.

A key lesson identified through this work is the value added by proactive engagement with the Supervising Engineer, particularly where the Undertaker may have limited familiarity with the Reservoirs Act or relevant engineering principles. In this case, CCBC benefitted from an approach whereby the Supervising Engineer provided constructive support beyond minimum compliance expectations, including clearly explaining how statutory requirements could be achieved in practice and undertaking thorough reviews of submitted information. Furthermore, since the involvement of the Engineering Projects Group in supporting the Undertaker, the delivery of works has become more efficient and coordinated, resulting in a smoother process and improved overall compliance. This collaborative, assurance-led approach has enhanced the robustness of submissions, supported undertakers in meeting their obligations, and reduced the risk of non-compliance arising from gaps in knowledge or experience.

Section 11 records and monitoring

A review of data and records was carried out and a few items where improvements could be made were evident. This included the taking of water levels and other instrumentation readings and recording them in electronic format for ease of processing and reviewing.

The Prescribed Form of Record was updated to the current format and made electronic. Readings are now reviewed by in-house engineers so any changes are picked up early.

Conclusion

We as a reservoir community need to help raise awareness and standards for our small Undertakers. Often their only contact with the profession is when the SE turns up for the S12 visit. If we can educate the Undertaker on their duties and requirements and help them by producing check sheets and record documents, and not just identify breaches in S12s or S10s but give the Undertaker the tools to be compliant we can raise the standard of small Undertakers to ensure their reservoirs remain safe.

REFERENCES

- Balmforth D (2021) *Independent Reservoir Safety Review Report, part B* (2020). Available at: <https://www.gov.uk/government/publications/reservoir-review-part-b-2020>
- HMG (2013). *The Reservoirs Act 1975 (Capacity, Registration, Prescribed Forms, etc.) (England) Regulations 2013*. The Stationery Office, London, UK, Statutory Instrument 2013 No. 1677.
- HMG (2022). *Reservoir supervising engineers: written statements and site visit reports*. Available at: <https://www.gov.uk/guidance/reservoir-supervising-engineers-written-statements-and-site-visit-reports>



The British Dam Society

Adaption and Improvements to Leigh Flood Storage Reservoir

CHRIS GROGAN, JBA Consulting
SAMANTHA HOWE, Environment Agency
IAIN PHILLIPS, Environment Agency
PAUL GRIST, KGAL Consulting Engineers Ltd.

SYNOPSIS

The Leigh Flood Storage Reservoir is a strategically important flood risk management asset in south-east England, reducing flood risk to 1,800 homes and 575 businesses in Tonbridge and Hildenborough. When a major improvement scheme was developed to increase resilience and expand capacity, the project presented significant design, planning and delivery challenges.

This paper outlines the development and delivery of resilient solutions for the Leigh FSR Expansion and Hildenborough Embankment Scheme, including how the asset was kept operational while new control gates were installed within the existing structure.

INTRODUCTION

Leigh Flood Storage Reservoir (Leigh FSR) was built after the 1968 floods, with construction completed in 1982. Located 2km upstream of Tonbridge, it is an online flood storage reservoir with an original capacity of 5.56 million cubic metres, regulated under the Reservoirs Act 1975. The embankment is up to 5.7m high and 1.3km long, and flows are controlled by three radial gates.

The FSR stores peak flows to reduce flood risk downstream in Tonbridge and Hildenborough, working alongside local flood defences and floodplain storage.

The River Medway Flood Relief Act (RMFRA) 1976 set the maximum legal storage level at 28.05mAOD and defined the conditions under which the reservoir can operate.

The scheme was originally built by the Southern Water Authority and is now owned and operated by the Environment Agency. In normal flows, the gates operate automatically and the storage reservoir is kept empty.

Adaption and Improvements to Leigh Flood Storage Reservoir

During high flows, trained operators manually control the gates using forecasting tools, live rainfall and river data, and agreed event plans that are reviewed and updated throughout the flood.

This operation optimises downstream protection by storing water around the flood peak and, in smaller events, can prevent impacts to properties altogether.

In more severe events, not all peak flows can be stored, so downstream flooding may still occur. The operating objective then is to minimise downstream flows while maintaining the safety of the structure and embankments.

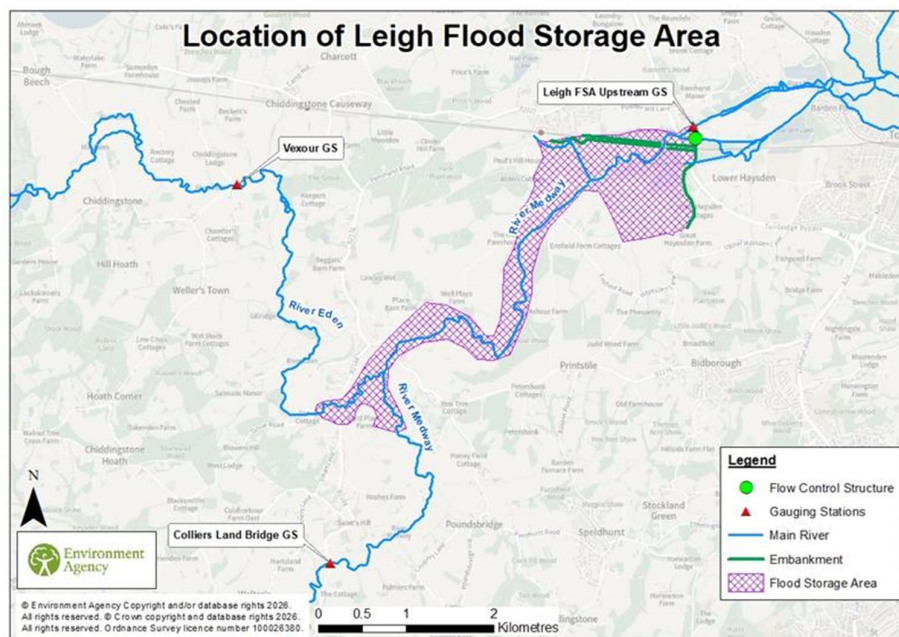


Figure 1. Location of Leigh FSR and gauging stations.

2013 FLOODS AND DEVELOPMENT OF LEHES

Since construction, the FSR has been operated 75 times. Procedures have evolved so that water is now stored later in an event, at flows above $75\text{m}^3/\text{s}$ rather than $35\text{m}^3/\text{s}$, reflecting improved understanding of the catchment and downstream capacity.

Two major catchment-wide floods stand out since 1982: October 2000 and December 2013. In both cases, the reservoir was fully used, but the severity of the events still led to flooding downstream.

The December 2013 event, on Christmas Eve, produced the highest recorded flows in the Upper Medway and the largest inflows into Leigh FSR, following 76mm of rain in 24 hours on an already saturated catchment.

Peak flow at Colliers Land Bridge reached $128\text{m}^3/\text{s}$, while peak inflow to the FSR was estimated at $250\text{m}^3/\text{s}$ from gauged data and $352\text{m}^3/\text{s}$ on the 15-minute operational assessment.

The storage area reached full capacity, with the reservoir level within 1cm of the legal Top Water Level. The operation of Leigh FSR limited maximum downstream outflow to $161\text{m}^3/\text{s}$, but the scale of the event still caused flooding in Hildenborough and Tonbridge.

A planned mechanical refurbishment was accelerated after the 2013 floods and developed into the Leigh Expansion and Hildenborough Embankments Scheme (LEHES), which will reduce flood risk to 1,870 homes and 575 non-residential properties through increased storage, extended asset life and separate protection works at Hildenborough.

- Increasing storage at Leigh FSR by 24% to $7.3\text{Mm}^3/\text{s}$ by raising the legal impounding level (TWL) from 28.05mAOD to 28.60mAOD
- Increasing the operational lifespan of Leigh FSR
- Deliver a separate scheme to protect Hildenborough.

The scheme aligned with the Catchment Flood Management Plan and the Middle Medway Strategy. Increasing the legal impounding level required amendment of the RMFRA, ministerial approval and a public inquiry in 2021.



Figure 2. Peak water impounded upstream of the control structure during the December 2013 floods.

DESIGN DEVELOPMENT AND PLANNING

JBA Consulting, supported by KGAL, developed the detailed design from 2019 with early contractor involvement from Volker Stevin. The strategy first addressed spillway resilience required under Measures in the Interest of

Adaption and Improvements to Leigh Flood Storage Reservoir

Safety, then increased reservoir capacity and delivered downstream flood protection works at Hildenborough.

Spillway resilience works were completed between 2021 and 2022, followed by the expansion works from 2022 to 2026. Planning conditions required the Hildenborough assets to remain non-operational until the expansion scheme was complete so that no properties experienced increased flood risk.

Delivery was also seasonally constrained, with winter access limited to preserve reservoir operational availability and summer water levels managed to avoid ecological effects.

Spillway resilience improvements

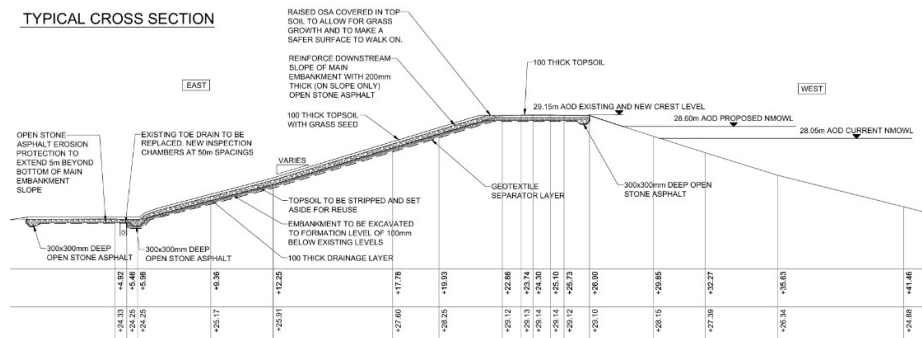
The Section 10 inspection in 2016 found that the existing erosion resistance of the embankments was inadequate for the overtopping velocities predicted by a flood study from 2010.

A further flood study was recommended to confirm whether the key design and safety check events could be passed safely, with a QCE to define timescales for any works. These recommendations formed a MIOS.

The 2018 flood study found overtopping velocities of 7m/s to 8m/s on the northern and middle embankments during the summer PMF event, and around 5m/s on the southern embankment because of elevated tailwater.

Initial proposals included articulated concrete block system for the northern and middle embankments and an open mat geotextile system for the southern embankment. The MIOS was completed in 2018 and the QCE required the erosion protection works to be completed within five years.

Design development then combined erosion protection with the capacity increase works. Because the proposed higher top water level exceeded parts of the existing clay core crest, the core was raised and regularised using a bentonite cement cut-off.



Following optioneering, open stone asphalt (OSA) was selected for the northern and middle embankments because it provided the required erosion resistance with lower carbon and cost than concrete mattresses. The southern embankment retained an open mat geotextile system, and the downstream toe drainage was replaced to improve performance.

Site works by the Contractor began in April 2021, and the MIOS was completed in December 2022. The use of OSA was referenced in Penman *et al*, (2024).



Figure 4. Installing OSA between the control structure and the railway in the north spillway section.

Leigh FSR expansion

The Leigh Expansion scheme reviewed the long-term effectiveness of the existing control structure and developed improvements to provide resilience over the next 60 years. The principal change was to raise the three radial gates by 600mm, increasing the top water level from 28.05mAOD to 28.6mAOD, with associated embankment raising, a short wave wall and a retaining wall in constrained areas on northside of the reservoir.

Key changes at the control structure included raising the gate shields, replacing the gate arms and pivot assemblies, introducing a drum-and-rope drive system, renewing the motor control and electrical systems, increasing operational redundancy, and updating the operator tools for the new arrangement.

Other improvements included a pumped eel pass to meet the Eel Regulations 2009 and wider landscape and ecology works to support the Water Framework Directive and biodiversity objectives.

Early lessons learnt

Early lessons from a comparable UK scheme, where gearbox issues emerged at factory testing, prompted a review of the gate drive philosophy.

Adaption and Improvements to Leigh Flood Storage Reservoir

The revised approach improved resilience and controllability through changes to the drive arrangement, motor and gearbox selection, torque sensing and cold-weather protection. It also affected the programme, so the wider delivery strategy was adjusted to protect the required in-service date.

Mechanical design approach & innovation

The replacement gates were developed around a resilient modern drive system that could accommodate civil misalignment while maintaining reliable operation and practical maintenance access.

Drive system selection logic

A study of leadscrew, hydraulic and winch-rope systems led to the selection of a standalone winch arrangement. It could be manufactured and tested off-site, then installed after gate replacement, reducing critical-path site work and avoiding extensive civil realignment.

Innovation in rope technology

Dyneema ropes were specified to provide a 25-year design life with low maintenance, reducing replacement frequency when compared with conventional steel wire ropes.

Torque monitoring and protection

Drive-train protection is provided by electronic torque monitoring using strain gauges and load cells, avoiding reliance on mechanical shear-key arrangements and allowing the system to detect over-torque conditions in service.

Gate structure and pivot design evolution

Spherical pivot bearings replaced the original cylindrical bushes so that long-term misalignment and pier tilt could be accommodated without major civil intervention. The new gate structures were then verified by Finite Element Analysis and first-principles checks for the higher hydraulic loads and overtopping conditions.

Maintenance access innovation

A bespoke rolling access platform was developed to allow downstream winch inspection and maintenance without routine use of harnesses or cranes, while also supporting lifting operations of up to 300kg by davit arm.

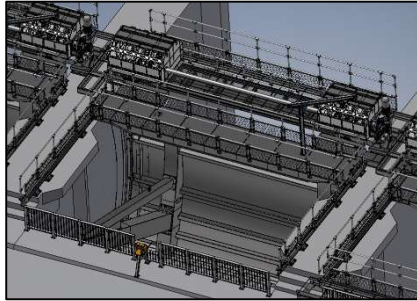


Figure 5. Moving platform in maintenance position

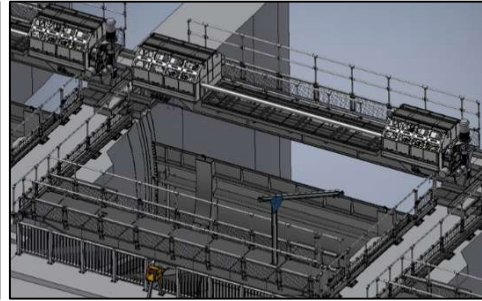


Figure 6. Moving platform in operational position.

Strategy change

Although the drive system change avoided abortive cost, it risked adding around a year to the programme. To recover time, the contractor proposed full gate replacement rather than in situ modification, allowing two gates to be delivered within a single season despite the site constraints.

The revised strategy extended the life of the M&E works, shifted fabrication into a controlled workshop condition, reduced site safety and pollution risks, and limited operational disruption. It also introduced major delivery challenges, including redesign to suit the as-found geometry, transport of oversized gate components, provision of a 300t crane on weak ground, controlled on-site welding and coating, and the design of stoplogs that became critical to reservoir safety during the works. For example, the gate shields were delivered to site in two parts due to height, then positionally set, welded and joint painted on site on a bespoke jig in large tent.



Figure 7. Low flow stoplog weir.



Figure 8. Gate assembly jig and scaffold for enclosure on site.

IMPLEMENTATION AND CONSTRUCTION CHALLENGES

Installation required a bespoke approach to accommodate the global movement of the existing piers and ensure asset continuity.

Adaption and Improvements to Leigh Flood Storage Reservoir

High-Precision Surveying Constraints

Detailed surveys were completed after the removal of the original gates, when the concealed side seal plates and pivot joints could finally be measured. These confirmed that the structure differed materially from the original drawings, with non-plumb walls, mismatched pivot levels and non-parallel seal plates.



Figure 9. 3D laser scan & photogrammetry survey.

Structural adaptation for pier tilt

The design responded to outward pier tilt by using sliding pivots that allowed controlled lateral movement without major reconstruction, although local concrete repairs were still required.

Stoplog integration and future maintenance

Temporary stoplogs were provided to protect the downstream community during gate removal. This allowed one gate at a time to be replaced in the dry while the other two gates kept the asset operational. The stoplogs provide protection up to TWL, due to works duration. The system was developed so that key components could be reused across the three openings and later retained to support future maintenance and commissioning tests.

OPERATIONAL CHALLENGES

There were numerous challenges to overcome during the 3-year construction period. Leigh FSR had to be kept operational at all times whilst the works took place, and 24/7 access was maintained for operational staff.

The works were carried out with the River Medway still passing through the structure and with upstream and downstream levels needing to be maintained. Gates were therefore replaced one at a time, with flows

managed through the remaining two openings and supporting procedures put in place to protect upstream and downstream interests.



Figure 10. Aerial view of original construction in 1982.

Safe access arrangements were agreed with the contractor and tested through joint exercises with the operational team.

Before gate replacement began, contingency measures such as the mobile generator and actuator were tested to confirm that the gates could still be operated if site power was lost.

Operation tools during construction

To operate Leigh FSA, the operators use a bespoke set of operational tools; Reservoir Balance Sheet (RBS) and Gate Opening Calculator (GOC). Data from the Medway catchment flood forecasting model is input into the RBS to identify onset of impounding and maximum reservoir level and outflow.

During construction the operators used revised RBS and GOC, supported by forecast and live catchment data, to plan and manage flood events. These tools were adapted for two-gate operation so that impoundment could be safely managed during each stage of gate replacement.

Two-gate operation identified some unavoidable early impounding because of differing weir cill levels. This was addressed through updated procedures, operator training, training exercises with contractor and site familiarisation with the new gates.

Adaption and Improvements to Leigh Flood Storage Reservoir

Low flow weir

When the centre gate was replaced, a low flow weir was developed to maintain upstream levels for landowners. CFD modelling was used to confirm its hydraulic performance, and contingency arrangements were prepared so that it could be removed quickly if a major flood was forecast.

Additional contingency procedures, including emergency stoplog removal, were developed and tested with operational staff during exercises.

Testing and commissioning

Testing and commissioning were completed in phases as each gate came into service. A new SCADA system was developed offline so that automation and manual control could be tested before live cutover, with the south and centre gates successfully operating through winter 2025/26 on the new system.

IMPROVED RESILIENCE OF FLOOD STORAGE RESERVOIR

The scheme improved resilience across the spillway, gates, controls and power systems. Measures included robust overtopping protection, dual motors, rope redundancy, duplicated motor control centres, automatic and manual operating modes, multiple power supplies with a generator backup, battery-operated last-resort actuation, and improved water level monitoring for operator verification.

Maintenance access and lifting was also planned as part of the resilience strategy, with arrangements ranging from manual handling of small items through to davit lifting, bridge-deck crane access and a permanent crane pad for heavy lifts such as stoplogs.

CONCLUSION

Leigh FSR has protected Tonbridge and Hildenborough since 1982, including during major events such as December 2013. The Leigh Expansion and Hildenborough Embankment Scheme has applied modern materials, systems and delivery methods to improve reservoir safety, increase resilience and maintain protection for downstream communities. It has also reinforced the value of good asset records to support future adaptation and renewal.



Figure 11. South gate leaf installed by a 300t crane.

REFERENCES

- Environment Agency. (2015). *Kent and South London Winter 2013/14 Floods - Medway Catchment*. Environment Agency, Bristol, UK
- JBA Consulting. (2015). *Severity analysis of the December 2013 flood, Medway Catchment*.
- Penman J G, Haywood S A, Wildee A L, Henderson N A and Smith R C (2024). Design and construction of an open stone asphalt spillway for Wychall Flood Storage Reservoir. In *Managing Risks for Dams and Reservoirs - Proceedings of the British Dam Society 2024 Conference*, British Dam Society, London, UK



The British Dam Society

Investigation and management of the 2024 Simpson overflow incident

TONY WOOD, Anglian Water Services Ltd

IAN KIRKPATRICK, Anglian Water Services Ltd

JOHN FOSTER, Mott MacDonald Ltd

SYNOPSIS

In September 2024 Simpson Reservoir, a rarely impounded flood storage reservoir constructed in the early 1970s, experienced sustained filling and overflow following prolonged heavy rainfall. The reservoir had not overflowed for at least twelve years. Although the overflow occurred as designed on 24 September, observed drawdown was significantly slower than predicted once inflows reduced, prompting investigation. Initial intervention focused on clearing the low-level outlet inlet screen and installing temporary siphons to assist drawdown, indicating underperformance of the 586mm diameter outlet culvert. As reservoir levels reduced, anomalous behaviour including a localised whirlpool and formation of a swallow hole was observed within the reservoir basin downstream of the outlet structure. These indicators suggested a subsurface hydraulic connection.

Further inspection, including diver intervention and CCTV survey following controlled drawdown, confirmed deterioration of the outlet conduit and a credible internal erosion mechanism along the conduit–embankment interface. The incident was escalated under the Reservoirs Act, with early involvement of the Supervising Engineer (SupE), an all-Reservoirs Panel Engineer (ARPE), and emergency planning teams (the regulator was also notified for drawdown). This paper describes the incident chronology, investigations, emergency risk management and remedial works, and presents transferable lessons for the management of normally dry flood storage reservoirs, with emphasis on behaviour led engineering judgement and outlet resilience during rare extreme events.

INTRODUCTION

Simpson Reservoir is a flood storage reservoir constructed in 1972–73 as part of a wider flood alleviation scheme associated with the development of Milton Keynes. The reservoir is designed to remain dry or only lightly impounded under normal catchment conditions, filling only during extreme rainfall events when inflows exceed the capacity of the low-level outlet. Five

Investigation and management of the 2024 Simpson overflow incident

permanent ponds were constructed within the reservoir basin in 2004, increasing total storage capacity and providing amenity features (a Construction Engineer was employed to oversee the alteration works).

The most recent statutory inspection under Section 10 of the Reservoirs Act was completed in March 2021. This inspection concluded that the embankment and appurtenant structures were satisfactory, the overflow capacity was adequate, and the outlet could achieve drawdown rates of approximately 2m per day when operating as intended. Internal erosion was not identified as a likely failure mode considering the 35m wide crest (the hydraulic gradient would be low), and the short duration and infrequency of impoundment.

The 2024 overflow incident therefore represents a case where observed reservoir behaviour diverged materially from prior inspection assumptions during an extreme but foreseeable operating condition.

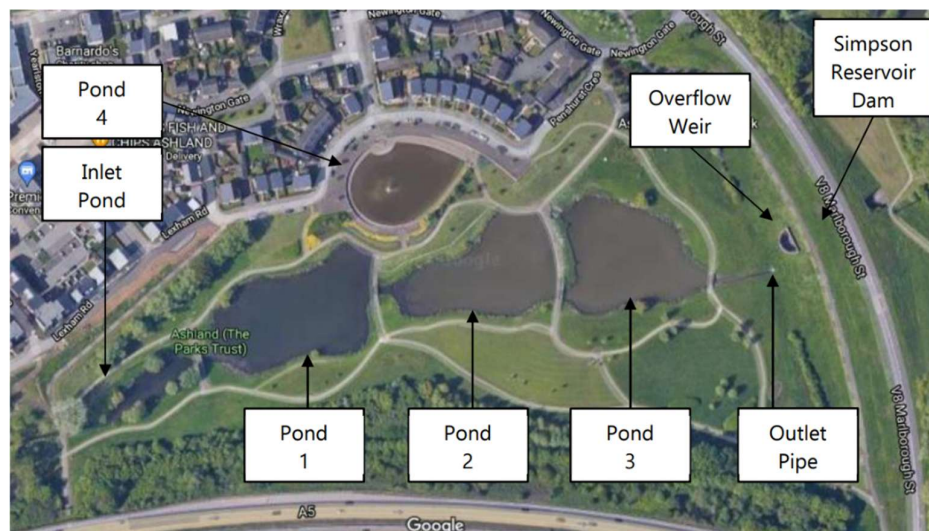


Figure 1 – Simpson reservoir overview

INCIDENT DESCRIPTION AND CHRONOLOGY

The reservoir began to fill on 23 September 2024 following prolonged rainfall across the catchment and overflowed on 24 September. Whilst overflowing occurred in accordance with design intent, drawdown was significantly slower than anticipated once inflows reduced.

On 26 September 24 the outlet screen was jetted at high pressure to remove accumulated vegetation and debris. This intervention resulted in only limited improvement and discoloured discharge was observed at the outlet. Temporary siphons were installed on 3 October to assist drawdown and supplement the low-level outlet.

Reservoir levels reduced gradually but with a localised whirlpool above the alignment of the outlet, and by 14 October had fallen approximately 2.45m below the overflow crest. At this stage a swallow hole approximately 0.8m by 0.8m in plan and around 1.5m deep was observed within the reservoir basin. The feature was located approximately 3.5m upstream of the overflow structure and aligned with the low-level outlet conduit. The geometry and position of the swallow hole prompted escalation of the event as a reservoir safety incident and the subsequent drawdown of the reservoir using over-pumping in accordance with the onsite plan.

Table 1 - Chronology of the investigation and management of the Simpson Reservoir overflow incident (2024–2026)

Ref	Date	Description
1	September 23/24 2024	Extreme Rainfall and First Overtopping
2	September 26 2024	Outlet jetted, negligible improvement, discoloured flows observed
3	October 3 2024	Prolonged impoundment, temporary siphons installed
4	October 7 2024	Re-install temporary siphons after vandals removed them. Whirlpool first observed
5	October 14 2024	Sinkhole observed, SupE and ARPE attend site, decision to drawdown by over-pumping (EA notified), emergency planning team notified, stakeholder meeting following SupE and ARPE assessment
6	October 16 2024	Emergency drawdown commences following pump arrival on site.
7	October 18 2024	Intrusive Investigations, Penstock confirmed fully closed, Divers deployed to clear grass and debris from outlet screen, CCTV survey confirms outlet conduit deterioration and internal erosion pathway.
8	December 2024	Temporary upstream debris screen installed, remediation completed, outlet conduit lined, sinkhole excavated and temporarily reinstated, behaviour and drawdown restored.
9	March 2025	Sinkhole excavated and permanently reinstated
10	July 2026 (Planned)	Long-term resilience Improvement, construction of permanent designed outlet screen to commence.

Investigation and management of the 2024 Simpson overflow incident

RESERVOIR LAYOUT AND CRITICAL FEATURES

The reservoir is formed by an earth embankment incorporating a central clay core and flanking filter zones. The dam crest supports a public cycleway and an adjacent road embankment. There is no permanent instrumentation installed, reflecting the normally dry operating regime and reliance on visual surveillance during rare impoundment events.

The low-level outlet comprises a 586mm diameter conduit with an upstream penstock that is normally maintained in the open position. The outlet discharges downstream beneath an adjacent canal embankment. A single overflow weir provides flood discharge capacity. Safe operation of the reservoir during impoundment is critically dependent on reliable outlet performance to limit both peak water level and duration of storage.

The previous Inspecting Engineer classified the reservoir as Category A, indicating a high consequence of failure. A Tier 1 Reservoir Risk Assessment was carried out in August 2024, where the assessment identified deterioration of culverts and internal erosion along structural interfaces as credible failure modes, but with low assessed likelihood due to the assumed infrequent operation of the reservoir.



Figure 2 – Reservoir at TWL



Figure 3 – Over-pumping

REGULATORY AND GOVERNANCE CONTEXT

Governance arrangements were reviewed as soon as reservoir behaviour departed from that predicted for normal flood storage operation. Prolonged impoundment, slower-than-expected drawdown, and the observation of a localised whirlpool at the reservoir surface directly above the low-level

outlet alignment were behavioural indicators that raised concerns that a focused subsurface flow path could be developing. The Supervising Engineer (SupE) Ian Kirkpatrick (Anglian Water) was informed promptly and remained actively involved throughout the evolving situation supported by Tony Wood (Reservoir Safety Engineer, Anglian Water). The SupE also notified the previous Inspecting Engineer, and a more local ARPE, John Foster (Mott MacDonald) who could attend site within two hours.

The ARPE advised that an active draw down of the reservoir was recommended, initially using temporary siphons and later by over-pumping, to reduce load on the embankment and provide safer conditions for investigation. Following this decision to draw down, the Environment Agency was formally informed, in line with statutory notification requirements. The notification explained the abnormal reservoir behaviour, and the rationale for undertaking controlled emergency drawdown as a precautionary risk-reduction measure.

In parallel, the situation was formally escalated within Anglian Water as an incident, in accordance with the company's emergency planning procedures and the site-specific Onsite Reservoir Plan for Simpson Lake. This internal escalation established a clear incident management structure, activated defined alert states, and formalised command, control, and communication arrangements. It ensured that decision-making responsibilities, escalation thresholds, and interfaces between reservoir safety, operational teams, and emergency planning were clearly defined should conditions worsen.

As discussed, the escalation was reinforced with an ARPE engaged alongside the Supervising Engineer to provide independent advice on the developing failure mechanism, investigation strategy, drawdown approach, and proposed remedial measures. This governance framework ensured that regulatory engagement, professional reservoir safety oversight, and emergency planning arrangements operated coherently and proportionately throughout the incident.

Investigation and management of the 2024 Simpson overflow incident



Figure 4 – Whirlpool above outlet alignment



Figure 5 – Sinkhole at location of whirlpool

SITE INSPECTIONS AND INVESTIGATIONS

A multi-disciplinary site inspection was undertaken on 14 October 2024 involving reservoir safety engineers, operational staff, maintenance teams, and contractors. It was noted that, prior to this inspection, a persistent whirlpool had been observed at the reservoir surface directly above the outlet alignment. Once reservoir levels fell below this elevation, the whirlpool ceased and the swallow hole became visible at precisely the same location.

This sequence of observations provided strong evidence of a concentrated subsurface flow path and informed the working hypothesis that the outlet conduit, rather than general embankment seepage or slope instability, represented the governing failure mechanism.

Emergency over-pumping was implemented under controlled conditions to reduce reservoir levels and hydraulic loading. Once it was confirmed that the upstream penstock was fully closed and hydraulically secure, specialist divers were deployed to inspect and clear the upstream outlet grill. Significant accumulations of grass cuttings and organic debris were removed, confirming that partial blockage had materially reduced outlet capacity.

With reservoir levels sufficiently reduced, a CCTV inspection of the low-level outlet conduit was undertaken. This confirmed deterioration at the crown of the conduit at the joint, providing definitive evidence of a credible internal

erosion mechanism along the conduit/embankment interface of fines entering the culvert, likely initiated by a pipe joint failure. This was consistent with the swallow hole and earlier hydraulic behaviour indicated by the whirlpool. A clear example of how quickly such a failure can lead to a pipe forming.

EMERGENCY AND INTERIM RISK MANAGEMENT

Emergency and interim risk management focused on rapid reduction of reservoir levels to remove load from the embankment and prevent further development of internal erosion. Measures included temporary siphons, emergency over-pumping, enhanced site surveillance, and continual review of weather forecasts.

Once the reservoir was drawn down, management actions were undertaken in accordance with the site-specific Onsite Reservoir Plan for Simpson Lake, which defines alert states, observable trigger conditions, command and control arrangements, and escalation pathways in the absence of permanent instrumentation. This provided a structured framework for decision-making, and coordination between engineering, operational, and emergency planning functions. In addition, a temporary upstream trash screen was installed, and the operational maintenance frequency increased.

A Flood Contingency Plan prepared during the incident supplemented the Onsite Reservoir Plan by defining green, amber, and red alert thresholds linked to reservoir level relative to the overflow and outlet structures. For each alert state, proportionate actions were set out, including enhanced inspections, outlet clearance, mobilisation of pumping, temporary ground protection using geotextile and stone, and preparation of downstream mitigation measures.

Immediately following identification of abnormal reservoir behaviour, the Anglian Water Emergency Planning team was informed by telephone. A Microsoft Teams meeting was convened shortly thereafter involving Emergency Planning, Reservoir Safety, and operational specialists. Once the Supervising Engineer and All Reservoirs Panel Engineer (ARPE) had assessed site conditions and confirmed that there was no immediate instability but an elevated vulnerability to further rainfall, a wider coordination meeting was held with key stakeholders to confirm incident status, objectives, and escalation thresholds.

Investigation and management of the 2024 Simpson overflow incident



Figure 6 - Sinkhole



Figure 7 – Outlet screen and sloping penstock

ENGINEERING DECISION-MAKING UNDER UNCERTAINTY

Engineering decision-making during the incident was undertaken in a context of limited direct information on the internal condition of the embankment and outlet conduit, with the reservoir operating outside its normal dry regime. Decisions were therefore guided by dam engineering principles, observed behaviour, and statutory engineering oversight rather than reliance on asset condition assumptions alone.

The Supervising Engineer and All Reservoirs Panel Engineer (ARPE) were central to this process from an early stage. Their involvement ensured that decisions were framed within recognised reservoir failure mechanisms and aligned with statutory reservoir safety duties. Initial emphasis was placed on understanding whether observed behaviours were consistent with credible internal erosion or piping scenarios, including along the interface between the outlet conduit and surrounding embankment fill.

Observed indicators including prolonged impoundment, reduced outlet capacity, discoloured discharge, an observed whirlpool within the reservoir basin, and later formation of a swallow hole, were collectively interpreted as behavioural symptoms consistent with subsurface seepage concentration and progressive void development. While no surface deformation of the crest or downstream face was evident, the nature and spatial alignment of these indicators suggested that a structural interface failure mode associated with the outlet conduit could not be discounted.

In this context, the ARPE advised that behaviour should be given precedence over likelihood scores derived from prior inspections or Tier 1 risk assessment outputs. This reflected established dam engineering practice that internal erosion mechanisms may evolve rapidly under sustained

hydraulic loading and may not present visible distress until late in the failure sequence. The Supervising Engineer concurred, emphasising that the duration of impoundment was extending beyond that assumed in the prior Section 10 inspection.

A conservative strategy was therefore adopted, based on the principle of reducing hydraulic load as the primary risk control measure. This led to decisions to:

- accelerate drawdown through siphoning and over-pumping,
- avoid intrusive investigation until reservoir levels were demonstrably reduced,
- secure the penstock prior to diver deployment, and
- treat the outlet conduit as potentially compromised until proven otherwise.

This approach deliberately prioritised risk reduction over diagnostic certainty. It reflected the judgement that continued loading during further rainfall events could materially increase the rate of internal erosion or lead to progressive collapse of the conduit crown, with potential loss of control of the reservoir.

Once drawdown had reduced reservoir levels and confirmed stability of the embankment, intrusive inspection was undertaken in a controlled manner. Subsequent CCTV evidence of conduit deterioration validated the earlier behavioural interpretation and reinforced the engineering rationale for the actions taken.

The incident demonstrates the value of early, active involvement of statutory engineers and the application of behaviour-led dam engineering judgement under uncertainty, particularly for structures that operate infrequently and therefore fall outside normal condition-based surveillance regimes.

PERMANENT REMEDIAL WORKS

Permanent remedial works (under an appointed QCE) were developed to address both the immediate failure mechanism and longer-term resilience of the outlet system. The primary structural intervention comprised full-length internal lining of the low-level outlet conduit to restore hydraulic integrity and eliminate the internal erosion pathway associated with deterioration at the conduit crown and joint. The swallow hole and surrounding affected ground were excavated and reinstated using suitable materials and method to restore continuity of the embankment fill.

Investigation and management of the 2024 Simpson overflow incident

In parallel, interim measures were implemented to reduce the likelihood of repeat outlet blockage. A temporary upstream trash screen was installed within the reservoir to intercept grass cuttings and floating vegetation before reaching the existing outlet grill, providing immediate filtration during subsequent runoff events.

The design for a permanent replacement outlet screen, in accordance with relevant CIRIA guidance on debris screening and hydraulic structures, has since been completed. The design provides increased hydraulic capacity, improved resistance to blockage by vegetation, and safer access for inspection and maintenance. Construction of the permanent screen is programmed to commence in July 2026, subject to final approvals.

LESSONS LEARNED

The incident highlights several transferable lessons:

- Rarely impounded flood storage reservoirs can experience significant internal deterioration during extreme events.
- Outlet condition and debris management are critical to limiting duration and consequences of impoundment.
- ALARP classifications require active scrutiny of governing failure mechanisms rather than reliance on qualitative likelihood alone.
- Behaviour-based indicators, particularly delayed drawdown, discoloured outlet discharge and localised whirlpools can provide an early warning well in advance of visible embankment distress.
- Clear onsite and flood contingency planning, combined with early engagement of emergency planning teams, materially improves incident response.
- AWS will review outlet screens at similar flood storage reservoirs and use targeted CCTV inspection of outlet conduits to improve assurance where impoundment is infrequent.
- Simple, improvised siphons formed from flexible pipe sections proved effective in supporting drawdown under suitable site conditions and can provide a practical interim control measure.
- Grounds maintenance activities can significantly influence flood storage outlet performance, and specific controls and awareness are required to prevent grass cuttings and vegetation increasing blockage risk.
- An existing contract to cover emergency reservoir responses proved a valuable factor in the rapid ARPE's attendance to site.

CONCLUSION

The 2024 Simpson Reservoir overflow incident illustrates how reservoirs that operate infrequently can nevertheless develop credible and potentially serious failure mechanisms when subjected to sustained hydraulic loading. Although the reservoir overflowed as designed, the prolonged duration of impoundment exposed vulnerabilities in the outlet system that were not evident during normal dry operation or routine visual inspections.

From a dam engineering perspective, the incident reinforces the importance of behaviour-based assessment as a primary indicator of risk. Trained personnel being present on-site during flood storage impoundment provided observations including delayed drawdown, outlet under-performance, discoloured discharge, a localised whirlpool within the reservoir basin, and subsequent development of a swallow hole collectively provided compelling evidence of abnormal internal behaviour. These indicators were consistent with a developing internal erosion mechanism concentrated along the interface between the outlet conduit and the embankment, despite the absence of visible embankment deformation.

Engineering decision-making was therefore driven not by prior condition assessments or qualitative likelihood scores, but by observed performance under load, in accordance with established dam safety principles. Early involvement of the Supervising Engineer and All Reservoirs Panel Engineer (ARPE) was critical in framing the response within credible reservoir failure modes and ensuring statutory duties were met. Their advice informed a precautionary strategy focused on rapid reduction of hydraulic load as the most effective risk control, recognising that internal erosion mechanisms can progress rapidly and may provide minimal visible warning prior to loss of control.

The decision to prioritise drawdown, delay intrusive investigation until reservoir levels were reduced, and treat the outlet as potentially compromised until proven otherwise reflected sound engineering judgement under uncertainty. Subsequent confirmation of outlet conduit deterioration validated this approach and demonstrated the value of conservative intervention when behaviour departs from design assumptions.

The incident also highlights the limitations of relying solely on infrequent statutory inspections and Tier 1 risk assessments for reservoirs that are normally dry. While such assessments appropriately identified internal erosion as credible and classified overall risk as ALARP, the event demonstrated how rare loading events can rapidly elevate risk beyond that implied by nominal likelihood classifications.

Investigation and management of the 2024 Simpson overflow incident

Overall, the Simpson Reservoir incident underscores the critical role of professional engineering judgement, statutory oversight, and behaviour-led decision-making in reservoir safety management. It provides a valuable case study for the dam engineering community, emphasising the need for outlet resilience, proactive response to abnormal behaviour, and robust emergency and contingency planning for flood storage reservoirs with low-frequency operating duty.

REFERENCES

- Environment Agency (2016). *Design, operation and adaptation of reservoirs for flood storage*. Environment Agency, Bristol, UK.
- CIRIA (2015). *Screening of water intakes and outfalls – guidance for debris management and hydraulic structures*. CIRIA Report C691, London.
- Institution of Civil Engineers (ICE) (2015). *Reservoir safety – Guide to the Reservoirs Act 1975. 2nd Edn*. ICE Publishing, London.
- International Commission on Large Dams (ICOLD) (2013). *Internal erosion in dams, dikes and their foundations*. ICOLD Bulletin 164, Paris.
- Anglian Water Services Ltd (AWS) (2024). *Simpson Flood Contingency Plan*, AWS, Peterborough, UK.
- Anglian Water Services Ltd (AWS) (2024). *Simpson Overflow Incident Overview*, AWS, Peterborough, UK.
- Anglian Water Services Ltd (AWS) (2022). *Simpson Onsite Reservoir Plan Simpson Lake*, AWS, Peterborough, UK.
- Anglian Water Services Ltd (AWS) (2024). *Risk Assessment of Simpson Balancing Reservoir – Tier 1 RARS Risk Assessment*, AWS, Peterborough, UK.
- Report on an inspection carried out under Section 10 of the Reservoirs Act 1975 Simpson Balancing Reservoir, July 2021.



The British Dam Society

Westmoor Reservoir: Lessons learned through design and construction

L. YUSTE ZABALLOS, AtkinsRéalis

T. SLATER, AtkinsRéalis

C. RESTORICK-VYSE, AtkinsRéalis

SYNOPSIS

Westmoor Reservoir is an off-line reservoir located within the Somerset Levels and Moors. The reservoir comprises three interconnected compartments, which together are classified as a single large raised reservoir under the Reservoirs Act 1975. There have been a number of Section 10 inspections, some of which have recommended measures to be taken in the interests of safety (MIOS). These have prompted the need for investigations and the subsequent delivery of improvement works. This primarily results from the application of modern legislation and guidance to legacy water management and drainage assets.

This paper discusses the lessons learned throughout both the design and construction phases of the project, with particular focus on the complications and risks associated with undertaking works within an operational reservoir subject to seasonal constraints.

Furthermore, the value of the collaboration between the client, designer, contractor, supervisor and specialist suppliers is discussed. This contributed to minimising technical queries (TQs), streamlining decision-making and improving constructability across the programme.

BACKGROUND

Situated in the Somerset Levels and Moors about 20km east of Taunton, Westmoor Reservoir lies within the Parrett and Isle catchments. The reservoir comprises three connected low-lying compartments: West Moor, Middle Moor and South Moor, collectively forming a single large raised reservoir under the Reservoirs Act 1975. The embankments defining these compartments were originally constructed in the mid-19th century under the Parrett Navigation and Westport Canal Act 1836 to support navigation and transport, and improve farmland by excluding frequent fluvial flooding, rather than to impound water.

The reservoir and its associated embankments are operated and maintained by the Environment Agency, who also maintain the principal hydraulic

Westmoor Reservoir: Lessons learned through design and construction

control structures (owned by the Environment Agency) and regulate water levels in coordination with the Parrett Internal Drainage Board. Seasonal management of levels in the Parrett and Isle catchment supports both agricultural practices, environmental designations and flood risk management, with elevated summer penning levels maintained for wet fencing and lower winter levels adopted to increase gravity drainage in the rhynes.

The historic river and canal embankments, combined with areas of natural high ground, create a washland reservoir that now provides approximately 8.5Mm³ of combined storage at the designated Top Water Levels across the three compartments.

Within the wider Somerset Moors system, Westmoor Reservoir interacts dynamically with adjacent rivers and canals through both controlled structures and natural exceedance routes. Under normal conditions, flows are transferred between compartments by the Westport Canal siphon and the Middelney siphon, with inflow regulated at the Slabgate Inlet and outflow largely managed via the Middelney Pumping Station. During flood events, the reservoir receives substantial additional inflows as rising water levels in the Rivers Parrett and Isle and along Westport Canal spill over low points of the adjacent embankments, including through Westmoor spillway (the Isle spillway).



Figure 1. Westmoor Reservoir under normal operating water levels.



Figure 2. Westmoor Reservoir under flood conditions.

Figure 3 provides a simplified overview of the main structures and flow pathways within Westmoor Reservoir, illustrating how water enters, moves through and leaves the reservoir. The sketch highlights the key control points alongside the primary spillways and locations where fluvial overtopping contributes additional inflow during flood events.

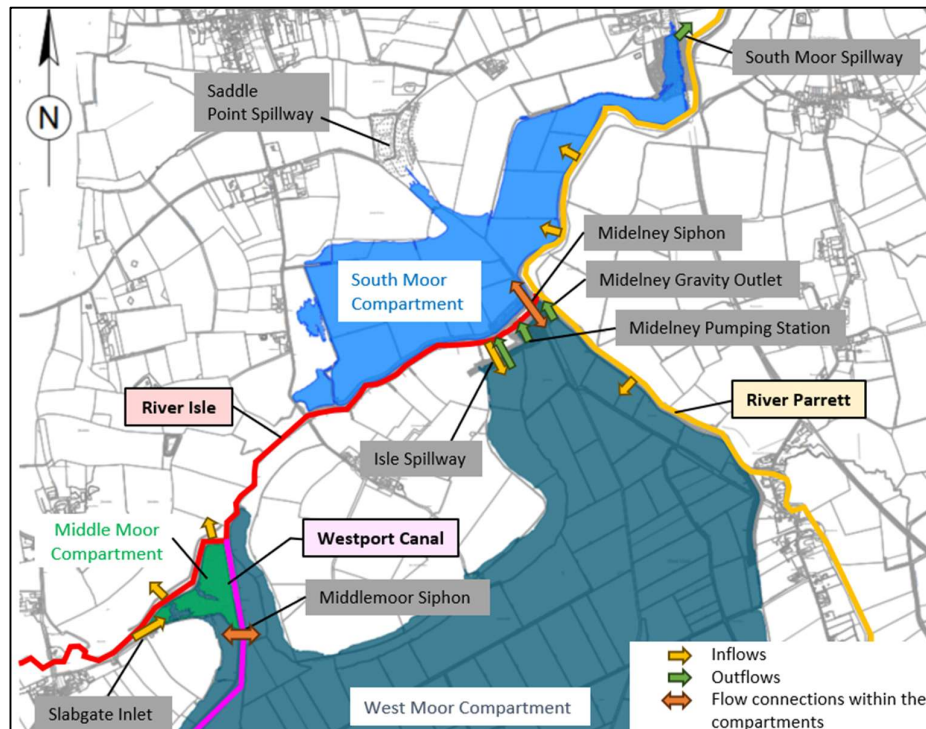


Figure 3. Site plan and key structures

MIOS RECOMMENDATIONS

A report on an inspection of the reservoir under Section 10 of the Reservoirs Act 1975 in 2019 stated that all the impounding structures were Category C as defined under Floods and Reservoir Safety 4th Edition (ICE, 2015). Measures to be taken in the Interest of Safety (MIOS) were recommended relating to the condition and performance of key hydraulic structures across the three compartments. The MIOS works involved the renovation or replacement of the gravity outlet and Middelney siphon, formalising the Isle Spillway and South Moor spillway, and producing a “Description of Works” to document the reservoir construction and function.

The MIOS works were delivered as part of the Environment Agency’s Collaborative Delivery Framework with AtkinsRéalis undertaking the design, Kier as the contractor, and Binnies providing construction supervision.

Construction commenced in 2023 and is ongoing, with all MIOS works programmed for completion in 2026.

Westmoor Reservoir: Lessons learned through design and construction

PROJECT SCOPE

Throughout the project, a greater understanding was developed of the reservoir risk and performance. This led to scope development while satisfying the MIOS.

The low spot on the heavily vegetated granular railway embankment in the South Moor compartment was highlighted as unsuitable for spilling flows due to the erosion potential. Accordingly, this low spot was replaced by a new grass covered embankment with a central concrete block spillway which is designed to safely pass spilling flows up to the 1 in 1,000 year event.

Local to the Middelney Pumping station, the scope included formalisation of the Isle Spillway to a defined top water level, to allow safe bidirectional flow between the River Isle and West Moor compartment.

Information and understanding from site investigations, assessments and studies was presented within the Description of Works document alongside reservoir flood mapping. The improved understanding of the risk posed to the public by the reservoir significantly changed the safety concerns and led to an early Section 10 inspection in 2024.

The 2024 report reduced the MIOS recommendations scope to construct the Isle Spillway and South Moor spillway only. Other non-statutory recommendations were made and included completion of the Middelney siphon replacement (which was being constructed at the time of issue) and replacement/renovation of the gravity outfall. Following this it was decided by the Environment Agency not to include the gravity outfall replacement/renovation in this scope of works.

The project sought to meet other Environment Agency performance outcomes while delivering the MIOS. These included the requirement for a “sustain” project, aiming to keep flood risk and water-level management unchanged. It was agreed with the Environment Agency that this would be achieved by maintaining the conveyance capacity of Middelney siphon. This was especially important given the complexity of the catchment and sensitivity to operational changes, which if not appropriately considered could lead to changes downstream that are difficult to assess. The project also aimed to meet Environment Agency operation and maintenance standards.

LESSONS LEARNED

The delivery of the MIOS works at Westmoor took place within a complex operational environment, where seasonal access constraints and programme pressures influenced both design and construction activities.

The lessons presented here aim to support decision making in future reservoir safety works within similar environments.

Site Constraints

The largest constraint on the project was aligning the fixed legal deadline combined with a limited and variable working window for construction due to seasonal flooding and environmental considerations. Typically, the reservoir is not flooded during half of the year between April/May and September/October. This constraint mirrors the etymology of the word “Somerset” which loosely translates to “summer settlers”. The limited and variable working window caused inefficiencies, as works at the back end of the summer needed to be timed to ensure completion before the site flooded. For example, in autumn 2023, the first spilling from the River Isle over the Isle Spillway into the West Moor compartment was on 3rd November, however in 2024 this occurred on 30th September.

The project experienced extreme weather, with 2023 being the hottest summer on record, and 2024 one of the wettest. The hot weather of 2023 caused desiccation cracking to the new South Moor embankment crest before grass had established. However this was found to be limited to the topsoil depth, which was infilled with a sand/topsoil mix. The variation in weather highlighted the importance of contingencies for extreme weather. Subsequently grass slopes were irrigated during prolonged hot weather.

Westmoor Reservoir’s grassed wetland is a habitat of international importance for overwintering and migratory birds with 2.1km² (38%) of the West Moor compartment designated as a SSSI. The reservoir also provides habitat to badgers, bats, otters, swans, fish and eel, and water vole. All of these have posed unique constraints to the works, requiring evaluation, design consideration and environmental mitigations in construction, which were agreed with relevant stakeholders and consenting bodies including Natural England and the Environment Agency Environmental Assessment and Sustainability team.

Collaboration between stakeholders

Throughout the MIOS works, close collaboration between the client, designer, contractor and construction supervisor was essential to maintaining progress and responding to emerging project requirements. Weekly coordination meetings were held either on site, in the office, or online depending on seasonal site constraints. These meetings provided a structured forum to review design development, include early contractor input, drawing on the team’s experience to address technical queries and

Westmoor Reservoir: Lessons learned through design and construction

site constraints as they arose. Shared collaborative team channels supported day to day communication and enabled prompt clarification of issues.

The strength of the team developed alongside the design. More face-to-face working and stronger relationships between people and organisations created safety, respect, and more constructive behaviours. This in turn encouraged open discussion of constraints and transparent consideration of construction risks, supported by a shared understanding of the MIOS deadlines and the programme pressures associated with working within a live reservoir system.

Of particular benefit for this project, the Environment Agency requested frequent on-site presence of the designer in addition to the construction supervisor. This approach reduced the number of technical queries raised and significantly shortened the time required for reviews and responses. It also allowed the team to adapt to changes in the scope more smoothly, as design adjustments and construction implications could be considered and agreed efficiently without delaying the works.

Spillway design and construction

The Isle Spillway is a 130 m long, two stage overflow embankment that provides a bidirectional hydraulic connection between the River Isle and Westmoor Reservoir. When water levels in the River Isle exceed the spillway crest, flows pass over the weir into the reservoir. As the river recedes, the spillway conveys water back towards the River Isle.

The spillway design was informed by hydraulic modelling undertaken using the Somerset Levels and Moors 1D model of the wider catchment. Spillway velocities were derived using normal-depth calculations, and erosion protection requirements were assessed using grass performance curves from CIRIA R116 (CIRIA, 1987). The calculations determined that a fully established average plain grass cover would be sufficient to prevent erosion due to overflowing for up to the 1 in 1000 year event. Nevertheless, prior to full establishment, the spillway would be susceptible to erosion from overflow. However, it was decided that a single season of grass establishment would be sufficient to resist erosion from the typical seasonal overflowing.

The construction programme significantly influenced early performance of the spillway. Delays associated with wet weather through the construction period, unexpected ground conditions and archaeological findings resulted in earthworks and installation of crest and toe beams being completed in early September 2024, rather than the originally planned date of July 2024.

Given the delay in the programme and late seeding, additional mitigation measures were considered such as coir matting or a lined low-flow channel. However, the risk of erosion due to overflowing was deemed tolerable.

On 30th September 2024, overflowing occurred across the 1V:10H reservoir side slope of the Isle Spillway. As the spillway had only been seeded in mid-September, the limited vegetation cover provided inadequate protection from erosion, which progressed rapidly leading to erosion of the topsoil and underlying embankment fill. As part of the planned contingency, the embankment was temporarily reinstated, which involved installation of heavy-duty plastic sheeting, weighted down with sand bags to allow the spillway to operate safely and reduce further deterioration throughout the winter period.



Figure 4. Isle Spillway erosion following the October 2024 events.



Figure 5. Temporary winter slope protection measures in operation.

The event highlighted the importance of considering the risks of programme delay during construction, particularly where seasonal flooding is expected. In addition, consideration should be given to providing additional mitigation during grass establishment, especially within complex catchments that are prone to flooding due to sequential rainfall events.

Due to the erosion that had occurred, the Environment Agency decided to consider alternative protection systems to enhance resilience during early life conditions. This was to reduce the risks associated with further overflowing of the spillway and subsequent delays to the project, and risk to MIOS delivery. A Dycel concrete block system was subsequently designed and installed to ensure the spillway could operate reliably prior to grass establishment and enable the MIOS requirement to be met ahead of the 2026 deadline, which would not have been certified with a plain grass cover until it had established.

Westmoor Reservoir: Lessons learned through design and construction



Figure 6. Isle Spillway reinforced with Dycel blocks in December 2025.



Figure 7. Isle Spillway in operation in winter 2025.

Siphon construction

Constructed in the 1830s, the Middelney siphon is an historically important brickwork inverted culvert under the River Isle which connects South Moor to West Moor. Sonar, CCTV and dive inspections identified mortar loss in the arch barrel of the culvert and indications of seepage into the siphon from above. To address concerns about the structural stability of the siphon and to ensure long-term capability of this structure, the Inspecting Engineer recommended the siphon to be renovated or replaced.

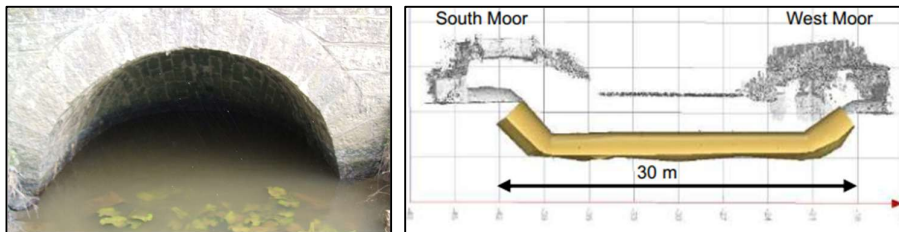


Figure 8. Middelney siphon outlet and 3D sonar survey

The Environment Agency required the siphon conveyance capacity to be preserved, to minimise change to flood risk and water level management. A reduced diameter liner that did not require person entry was discounted. Renovation of the siphon was discounted as the confined space risks associated with working within the siphon and person entry to the siphon were not considered acceptable. A project decision was made in 2022 to install a new siphon via tunnelling rather than install by cut and cover installation. This was due to programme risk, the spirit of the CDM Regulations and to enable safe operation, and the environmental constraints of in river works.

The siphon pipe was designed as 2 no. 220m long 1.4m dia. steel pipes tunnelled by direct pipe method under the River Isle. Under this method the pipe is pushed through the ground by a pipe thruster and a micro-tunnel

boring machine at the front of the pipe cuts through the ground. The tunnelling works required launch and reception pit cofferdams, and a large staging area for welding the pipe and feeding into the pipe thruster.



Figure 8. Pipe thruster



Figure 9. Micro-tunnel boring machine

The launch and reception pits were constructed in 2023, and the tunnelling works were completed in 2024. Reinforced concrete intake structures were built at the pipe ends in 2025 within the launch and reception pits. These intake structures connect to the existing moors drainage network with new drainage channels excavated in 2026.

The intake structures include stoplogs to allow inspection and maintenance of the siphon, including cleaning and dewatering of the pipeline via 'pigging', a method successfully adopted from the water pipeline industry where a foam 'pig' is driven through the pipe by compressed air or water.

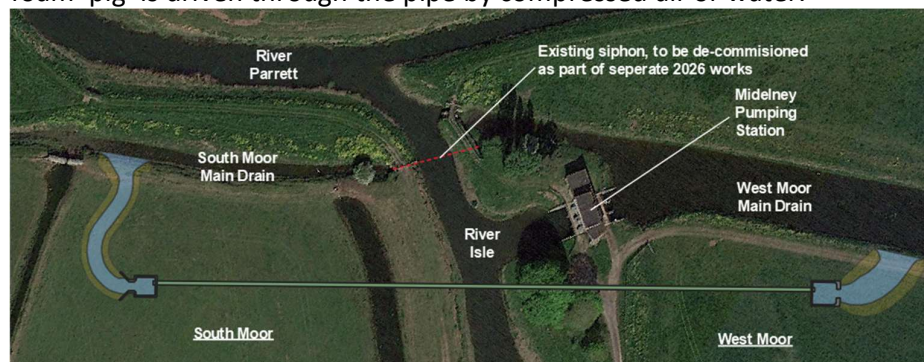


Figure 10. New Middelney siphon plan and long section

To satisfy the "sustain" project requirement the siphon pipes were designed as two 1.4m dia. pipes to replicate the capacity of the existing siphon. Due

Westmoor Reservoir: Lessons learned through design and construction

to the procurement delays with the supply of the pipes from overseas, and adverse unexpected ground conditions, tunnelling of the first siphon pipe was protracted and completed in October 2024, leaving insufficient time to complete the second pipe before the winter flooding season. Specialist tunnelling equipment was safely demobilised from site ahead of the winter floods.

The Environment Agency instructed AtkinsRéalis to investigate the impacts of having only one siphon pipe, in light of change in understanding of safety risk to the public. The study (Nov 2024) found replacing the existing siphon with one siphon pipe has a negligible impact on flood risk (despite reducing the conveyance capacity) and was not anticipated to affect water level management. Rather than remobilising tunnelling equipment from spring 2025 to tunnel the second pipe the Environment Agency decided to accept the single pipe only. The single pipe solution still satisfies the requirement to not change flood risk and maintains level of service.

When the decision was made to tunnel the siphon in 2022, the concept design and ECI informed costing estimate was based on a 110m siphon pipe, sufficient to span under the River Isle, banks and an adjacent drainage ditch. However, when the tunnelling contractor completed detailed design the siphon pipe in 2023 accounting for buildability, the length of the siphon pipe increased to 220m due to an increased cover allowance underneath the River Isle bed, and a revised angle of entry into the ground from the launch/reception pits and the minimum tunnelling bend radius of 1000m.



Figure 11. Temporary works – Pile layout area and plant

From concept to detailed design, the anticipated scale of the temporary works required grew significantly. For example, when fed into the launch pit the pipe was supported by 2 no. 100t crawler cranes. These cranes in turn required a 1m deep piling mat over the footprint of the pipe laydown area. Additionally, a temporary bridge was installed over the River Isle to allow plant access for recovery of the micro-tunnel boring machine.

It is noted that the overriding concern was with meeting the programme, due to the challenges associated with the seasonal working constraints and the MIOS deadline of September 2025. The accepted solution was difficult to achieve within the allowed time. Additionally, the delays and record poor weather conditions further reduced the available time on site. This highlights the importance of holistic reviews at key stages during a project and when significant change is identified. However, it is acknowledged that due to time pressures this would have been difficult to address at the time.

CONCLUSIONS

The Westmoor Reservoir scheme has highlighted the challenges of delivering reservoir safety improvements within a live system constrained by seasonal flooding, environmental factors and statutory MIOS deadlines. The project demonstrated the importance of aligning design assumptions with construction programme, particularly where performance depends on vegetation establishment. The Isle Spillway erosion event emphasised the need to consider programme uncertainty, the possibility of the typical seasonal floods being exceeded and the provision of mitigation measures during vegetation establishment.

The works also showed the value of revisiting key design decisions as new information becomes available or changes occur through construction. In particular, reassessment of the flood impact due to reduced siphon capacity demonstrated that simpler design solutions may be viable if the project requirements were revised at an earlier stage. However, this is challenging when faced with statutory deadlines and float eroded by poor weather and site constraints.

Notwithstanding the above, strong collaboration between all parties was key to delivery, with regular engagement and on-site presence enabling rapid decision-making and improved constructability. Overall, the project underlines the importance of managing programme risk, providing contingency during early operation, and fostering collaborative working to support efficient delivery of reservoir safety works.

Westmoor Reservoir: Lessons learned through design and construction

REFERENCES

CIRIA (1987). *Design of reinforced grass waterways (Report 116)*. Construction Industry Research and Information Association, London.

ICE (2015). *Floods and Reservoir Safety (4th ed.)*. ICE Publishing, London.

Met Office (2026). *Summer 2025 is the warmest on record for the UK*.
<https://www.metoffice.gov.uk/about-us/news-and-media/media-centre/weather-and-climate-news/2025/summer-2025-is-the-warmest-on-record-for-the-uk>



The British Dam Society

Cove Brook Flood Storage Reservoir – upgrading to meet modern reservoir safety standards

S BENN, Jacobs

P CHAPMAN, Jacobs

K ATTFIELD, Environment Agency

S CHENERY, BAM Nuttal

C BUCKLE, BAM Nuttal

SYNOPSIS

The Cove Brook Flood Storage Area (FSA) was constructed in 1974 and is retained by an earthfill embankment approximately 2.25m high and 900m long immediately upstream of a dense suburban area.

The latest Section 10 report made a recommendation in the interests of safety to assess the ability of the embankment to resist overflow arising from the 1,000 year and 10,000 year floods. The assessment showed that it would be disproportionate to meet the standards-based approach as described in Floods and Reservoirs Safety (FRS) (ICE, 2015). Therefore a risk-based approach was adopted, with the design standard varying with the consequences of failure in each section of the site.

The paper describes both the design and construction of the remedial works. The latter was not without its challenges, including earthworks challenges, working within a constricted linear site including several residents' gardens and dealing with protected animal and invasive plant species as well as public interactions where well-used public access into a nature reserve was blocked off.

INTRODUCTION

The Cove Brook Flood Storage Area (FSA) is situated on the western side of Farnborough and was constructed in 1974 as part of the Upper Cove improvement scheme to a design by Binnie & Partners for Thames Water. It is now operated and maintained by the Environment Agency (EA). The reservoir is retained by an homogenous earthfill embankment approximately 2.25m high and 900m long. The reservoir has a stated capacity of 95,000m³ and a surface area of 179,000m² when full to its original top water level at 62.63m AOD.

Flow on the Cove Brook is restricted by a 0.7m wide concrete flume with an adjacent spillway structure into the left bank consisting of an approximately 8m long primary spillway with a concrete slab over the crest and

Cove Brook Flood Storage Reservoir – upgrading to meet modern reservoir safety standards

downstream slope, and an approximately 40m long auxiliary spillway with a concrete slab over the crest and grassed downstream slope. The Cove Brook FSA is situated in public land and runs through nine private gardens.



Figure 1. View of original Cove Brook FSA embankment in public park

Following a Section 10 inspection in 2018 the Inspecting Engineer recommended a Measure in the Interests of Safety (MIOS) (Atkins, 2019) to carry out an investigation *“to assess the ability of the embankment to resist overflow arising from the 1,000 year and 10,000 year floods. The results of the investigation should be discussed with an All Reservoirs Panel Engineer (ARPE) to determine whether further studies into the 3,162 year return period flood and/or any works are required.”*

The flood category of the FSA had changed over the years, but the last Inspecting Engineer considered the FSA to fall within category A under FRS. However, based on the FSA being a small reservoir located on a relatively large catchment, he considered a 10,000-year design flood overly onerous. He agreed with the findings of a flood and dam break study undertaken in 2006 by Halcrow which proposed the design flood to lie between the 1,000 year and the 10,000 year return period events. The Inspecting Engineer concurred with the 3,162 year return period (the logarithmic mean between the 1,000 and 10,000 years) to be selected as the design flood. He concluded that *“on this basis, the dam should not be allowed to fail for floods up to 3,126 years. In larger floods the incremental damage attributable to a dam breach can be regarded as insignificant.”*

DESIGN PHASE

Assessment of embankment to resilience of overflow

A detailed hydraulic assessment was undertaken to assess the resilience of the embankment to overflow (Jacobs, 2021). Since the profile of the 900m long embankment varied significantly, the first step was to define the crest level and downstream slope gradient, by extracting data from existing EA survey information.

For the inflow, the findings from the 2006 Halcrow flood study were compared against flows derived using the rapid method in FRS (ICE, 2015). The latter were found to be lower and hence the 2006 results were used. The 3,162 year flood was not readily available from either method and was interpolated using the flows from the 2006 study (Figure 2).

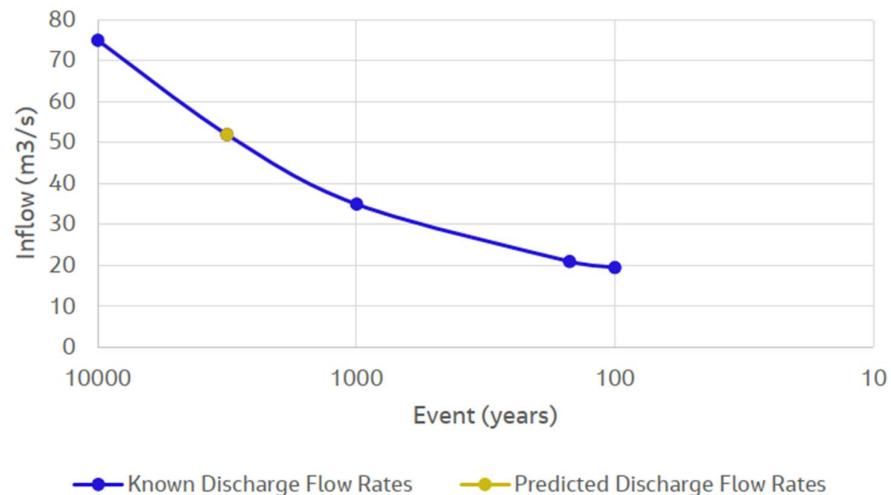


Figure 2. Interpolation of inflow for the 3,162 year design flood event

The derived flows were then used to assess overtopping depths using the broad-crested weir equation to estimate discharge across each section of embankment. Cross-sections were defined at 5m intervals along the crest, and the discharge for each section was calculated. The total discharge was obtained by summing the individual section flows, and a goal-seek approach was used to determine the water level required to pass the total event flow. Subsequently flow velocities on the downstream slope were calculated using Manning's equation for open channel flow. Conservatively it was assumed that terminal velocities would be reached.

Velocities were compared to limiting velocities for grass and reinforced waterways stated in CIRIA 116 to determine if the current embankment could withstand flows during the 3,162 year event. The existing downstream

Cove Brook Flood Storage Reservoir – upgrading to meet modern reservoir safety standards

slope was covered in grass, which varied between poor quality for the majority of the section in the public park and good quality in some areas.

The assessment indicated that the embankment would not be able to withstand the 3,162 year design flood without damage, except at the primary spillway. The primary spillway was predicted to overtop during the 5 year event, while the remainder of the embankment's performance suggested an annual exceedance probability equivalent to a return period of approximately 1-in-200 to 1-in-300 years.

Selected approach to increase embankment resilience to overflow

Achieving the standards-based approach in FRS would have required diverting the river to make space for a larger bund, buying land, building a higher bund in nine private gardens, plus increasing the number of culverts through the railway embankment 300m downstream to stop water backing up and flooding houses during the dam safety flood. These works were deemed disproportionate and hence a risk-based, section-specific approach was adopted. Individual sections of the embankment were assessed independently to define proportionate levels of overflow resilience, rather than applying a uniform standard across the asset.

Each section was evaluated based on the downstream consequences of potential overtopping or breach, including societal and annualised loss-of-life metrics. This approach effectively subdivided the embankment into sections with section-specific resilience requirements. It was deemed reasonable to retain the 3,162 year design flood for the main embankment in the park, but adopt a 10,000 year design flood for the section through the private gardens.

Following the assessment of the embankment to resilience of overflow the final design solution was selected (Jacobs, 2024). This essentially comprised lengthening the existing auxiliary spillway and lowering / regulating the embankment in the public park to create an emergency spillway. Non-overflow sections in the gardens were raised to provide more resilience to long-term settlement and low spots were addressed. The emergency spillway embankment in the park was moved upstream to achieve a flatter downstream slope and avoid an existing surface water sewer downstream. The new embankment was designed with a minimum 700mm deep key trench into the foundation and dedicated pedestrian crossing points were added. Details are shown in Figure 3 and Table 1.

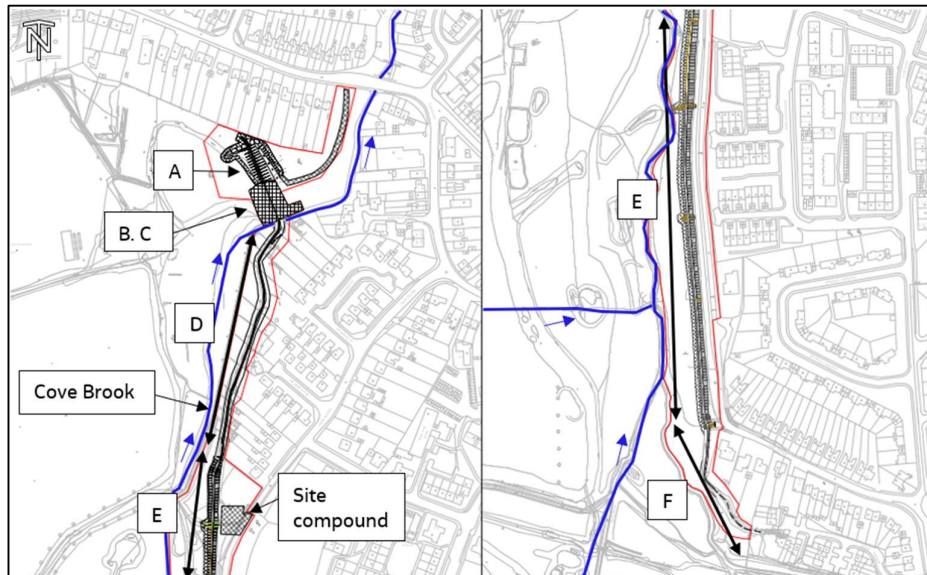


Figure 3. Cove Brook FSA overview plan (© Crown copyright 2023. Licence No. 100024198)

Table 1. Summary of selected option per section

Ref	Section	Existing	Description of works
A	Auxiliary Spillway (Cove Road)	40m length at 62.20m AOD	Lengthen by 20m and provide new toe channel.
B	Primary Spillway	7.78m length at 62.14m AOD	None
C	Flume	0.7m wide	None
D	Embankment north end (non-overflow through gardens) Ch 70-300	Crest level varies 62.58m AOD to 62.78m AOD	Required level 62.70m AOD. New crest kerb to 62.75m AOD to allow for future settlement.
E	Emergency Spillway (Hazel Avenue) Ch 300-790	Crest level varies; embankment not designated as emergency spillway	Create over-toppable bank with crest at 62.54m AOD and flatter downstream slope.
F	Embankment south end (non-overflow) Ch 790- 921	Crest level varies 62.58m AOD to 62.78m AOD	Required level 62.70m AOD. New crest kerb to 62.75m AOD to allow for future settlement.

Cove Brook Flood Storage Reservoir – upgrading to meet modern reservoir safety standards

ENABLING WORKS

Tree removal

The sections at Cove Road and along Hazel Avenue were heavily populated with unmanaged, self-seeded trees. To enable construction access and increase sunlight for future grass growth, several vegetation removal options were considered, ranging from full clearance down to the brook, to selectively removing only low value trees. A walkover assessment involving Park Rangers, the construction team, arboricultural specialists, surveyors, and designers established an efficient method for categorising and prioritising trees.

A 3m wide working corridor upstream of the bund was agreed to provide safe access for construction. All trees within this zone, including those with root protection areas heavily affected by haul roads and construction activities, were removed on safety grounds. High value trees on the construction boundary were retained and, where required, selectively coppiced. Additional areas for passing places and material storage were defined. More than 150 trees were removed in early 2024. The works were timed to avoid the birdnesting season.

Ground conditions / Ground model

Available ground investigation data and geological information included published geological information, site investigation information for the nearby Esso Southampton to London pipeline, and geotechnical and geo-environmental desk study data including UXO risk assessment. A project specific ground investigation (GI), with fieldwork was undertaken in 2023.

The 2023 GI included testing of the material in the existing bund, to assess if the existing fill trimmed from the downstream side of the bund could be re-used as embankment fill. From the results, it was identified that some of the existing fill could be reused in the bund. However, in discussion with the contractor, this was not deemed practicable, primarily due to the size of the existing bund making it difficult to separate top soil and other organic components to provide a suitable fill. As a result, it was decided to import clay fill for the construction of the emergency spillway embankment. Existing topsoil was retained as much as possible and reused.

Public engagement

The Cove Brook FSA is located through local authority owned land; Southwood Park, as well as nine residential back gardens. Therefore, careful consideration was required to engage with all stakeholders before, during and following the works, to ensure they were updated at all stages.

The engagement was made more challenging due to the recent re-laying of the Esso pipeline through Farnborough, which had already caused disruption to road and Southwood Park users, as well as the removal of many mature trees which had caused a lot of controversy throughout the local community. As mentioned above, the MIOS project required many trees to be removed, which contributed to the vast amounts of complaints received by the EA. Therefore, it was important for the public to understand the reasons behind the removal from an early stage.

The EA team posted over 200 letters to residents inviting them to a public engagement meeting at the local country park meeting room. This enabled the team to meet the local community and explain why the works were necessary, and how it would benefit the downstream community. Illustrations were displayed using before and after pictures of what the park would look like following the works. The team also set up a display on site over two days to capture other park users. Additionally, an information newsletter was produced and sent to local people, and on-site boards were installed to keep members of the public updated on the works. The newsletter was an important tool in answering the many queries that came through the EA's communications team, which saved time writing individual letters. It was also important to maintain the good relationship with the nine homeowners that had been built up over the previous four years. They were therefore informed of the works on a more personal basis, with regular check-ins and personalised letters and newsletters.

It was important to highlight the benefits that the project would provide; working closely with Rushmoor Borough Council to create an Environmental Package to compensate for the trees that had been removed; funding of £70k towards the Rushmoor Borough Council Cove Brook Channel Improvement Project for biodiversity benefits. Better flood resilience of the flood defence, and the expansion and access improvement to the footpath for wheelchairs and prams.

CONSTRUCTION CHALLENGES

Construction of the works commenced in April 2024 and concluded in Winter 2025. It was not without challenges, as described below.

Linear and constricted working site

Over its life the existing bund had been encroached upon by vegetation and trees resulting in a constrained linear construction area. It was required to keep tree clearance to a minimum, and this along with the relocation of the new bund created a difficult site in which to operate. In an ideal scenario

Cove Brook Flood Storage Reservoir – upgrading to meet modern reservoir safety standards

space would have been available to clear the existing bund whilst simultaneously constructing the replacement alongside.



Figure 4. View of section of old bund (left) and new embankment construction with additional key trench into foundation (right)

Additional constraints were the requirement to re-use topsoil which needed to be stockpiled until required, stockpiling and processing incoming clay prior to use and disposal operations for surplus material from the original bund.

To further complicate the site operation from a construction perspective, the need for large plant movements could generate a significant safety hazard. Safety or zonal working procedures for maintaining safe working conditions for operatives working around plant and machinery require that adequate segregation be established as a primary control. This is especially effective in controlling the risks due to blind spots.

To meet these conflicting requirements, instead of being able to simply start at one point and move through to the end, it was necessary to establish a number of separate locations for each activity. Sections of existing bund were cleared and temporary flood protection put in place to provide stockpiling and processing areas with materials transported along the temporary access road as required.

Clay source

Finding a suitable clay source was the first task upon commencement of the construction contract. Several quarries were approached, but it took several

months until a suitable source was confirmed, causing a significant delay to the project. The reasons for this were specific earthworks specification requirements and a slow turn-around of test results by the laboratory. Once a suitable clay was approved, the construction challenge presented was principally one of logistics.

The moisture condition value (MCV) versus moisture content (MC) relationship was used to control compaction of the clay in-situ. A target MCV range of 5.2 to 11.5 with optimum MC ranging between 21% and 25% had been agreed based on source testing. Given the site constraints, ideally the clay arriving on site would have been placed immediately ready for compaction. However, the clay usually arrived too dry and conditioning was required. The clay was spread and worked in situ whilst adding water using a bowser and jet wash. The addition of water was timed and after a few trials the operatives became a feel for how much water to add. To reach a usable stable condition the clay was left overnight before re-testing and compaction the next day.

External laboratory testing could take up to 10 days to turn around MC and MCV test results. To mitigate program delays a site testing regime was adopted. The clay was sampled on arrival and the samples prepared and dried using a microwave oven; duplicate samples were also provided to a UKAS certified laboratory. When the quick site testing indicated the correct MC the material was then placed and compacted with accurate notes taken of location. In addition site compaction testing was performed.

The site tests and independent laboratory tests mostly aligned, providing confidence that the final structure was within specification. Only in a few sections the external testing resulted in re-work.

With narrow constraints on the material during installation the project did become subject to adverse weather conditions and ultimately was interrupted for several months during which temporary protection was installed to control the clay condition.

Invasive species

Prior to start of construction the ecological assessment identified invasive species (variegated Yellow Archangel and rhododendron). To comply with the contractor's duties not to allow the spread of invasive species several measures were taken.

At the mobilisation of the site an ecologist was engaged to walk the site identifying the locations and types if invasive species present so these could be cordoned off. Affected areas were also equipped with a boot wash facility and plant was cleaned thoroughly before leaving the area.

Cove Brook Flood Storage Reservoir – upgrading to meet modern reservoir safety standards

All site supervisors completed invasive non-native species (INNS) training and all site personnel were briefed on the details of the specific invasive species and the need to control it correctly.

Where possible the located areas were left in situ and no further action was required other than periodic inspection that no spreading into the work area was taking place.

The actual incidence of invasive species in the work areas was low, and this allowed most activities to proceed without undue interruption. When an area of contamination was identified that required clearing then an ecologist was brought to site to advise on the operation. In some instances, this was hand grubbing the plants and otherwise sections of topsoil were removed, bagged and disposed at registered locations.

Working in resident's gardens and interaction with the public

To work freely and safely within resident's gardens, the EA's estates team sent a Notice of Intended Entry for the contractor to enter their land. The Notice of Entry not only allows access to carry out work, but also provides landowners with indemnities under the Act for any losses or damages, should any occur. Works in the gardens was prioritised; however it became apparent that not all works would be completed by the winter of 2024, and therefore Heras type fencing remained in place for longer than anticipated.

It was necessary to close public footpaths during the works to safeguard the public due to moving plant. This meant access into the park from the south was not possible from April 2024 to November 2024, although access from the north, from Cove Road, was maintained throughout the works. Many users of the park would normally access the park from the south, which caused some hostility towards the contractor, particularly as the park was closed during summer. This resulted in many complaints by email and phone, and to the contractor and EA staff on site. The updated newsletter produced at the time helped to manage the situation and answer many questions; however, it took time to respond and appease a small minority of consistent complainants. When the contractor temporarily demobilised over the winter 2024, some footpaths could be temporarily opened to the public. It was important to manage expectations at this point, and to make pedestrians aware that the footpaths would again be closed once works commenced in the Spring of 2025.

Temporary flood protection

Due to adverse weather conditions in November 2024, the contractor was forced to demobilise, however, the construction of the emergency spillway

was only partly finished. A temporary flood defence was required to protect properties over winter. The temporary flood barrier was designed by the contractor's temporary works team, and was a combination of bulk bags and polythene sheeting and a temporary bund established with stockpiled clay. The EA's A-frame type flood barrier was investigated as an option but deemed impractical due to lack of storage space, and complexity in installation due to the uneven ground on site.

Once constructed, the temporary defence was fenced off and the contractor was responsible for its integrity and regular inspection. It was agreed that for normal impounding operation, the contractor would carry out management inspections and actions to prevent seepage and compromise of their works, and would liaise closely with the EA's flood resilience team. A flood protocol was written up by the EA which included actions to be taken by whom at each telemetry alarm level.

It is worth adding that temporary flood protection could also be installed at incomplete sections of the embankment over the weekend if weather reports suggested risk of significant rain.



Figure 5. View of section of work with bulk bags ready for installation

Badger remedial works

Due to the location within a country park, wild animals were frequently encountered and several old fox burrows in the embankment were filled as part of the works. The southern embankment (chainage 800+) did not require any works because it was already significantly higher than the required levels. Therefore this section was fenced off to prevent public

Cove Brook Flood Storage Reservoir – upgrading to meet modern reservoir safety standards

access and was not often visited by the construction team. During a walkover in spring of the first year of construction, three new burrows were identified in this area. An ecological camera-trapping exercise was undertaken to determine which animals were using the burrows and to establish whether a protected species was present. Burrows were found on both sides of the embankment, raising concerns that they could link beneath the crest, creating a potential weak point and flow paths.

Camera footage confirmed that the burrows were used by multiple animals, including badgers, which is a protected species under UK law. Following further desk-top historic ecological investigations it was confirmed that the sett was not the main one, and as a result, an assessment and licence application to close the sett were progressed. Closure was carried out by licensed professionals during winter 2024, along with filling the burrows temporarily with clay bags and covering the area in mesh to prevent re-entry.

Following closure of the badger sett, remedial works were agreed. The most cost-effective solution involved filling the voids with a geosynthetic resin pumped into the sett. This material is considered virtually impermeable and has a high compressive strength once cured (150kPa).



Figure 6: Pressure injection of resin into centre of sett

The mesh was locally cut and the clay bags removed to open the entrance points to the sett. To confirm connectivity and better understand the size and shape of the sett, a CCTV camera survey was carried out. With the sett mapped out, the filling process involved injecting warm resin under

pressure, starting from the centre of the set using a resin injection lance. The resin was pumped into the sett until it emerged at each entrance.



Figure 7: Resin expanding to fill sett

A total of 18.5kg of resin, which expands between 10 to 15 times its liquid volume, was injected. This equates to a volume of approximately 0.5m³. Resin injection was carried out in one day. The entrances to the burrows were then covered with topsoil and to prevent future burrowing, the entire southern section of the embankment was covered with a badger-proof mesh. The mesh on the surface however presented a safety hazard for the public and grass maintenance due to it being slippery, and therefore a layer of topsoil was added to cover the mesh and allow the existing grass sward to grow through, creating a well-bound surface structure.

SUMMARY

The Cove Brook MIOS project has shown how an existing flood defence asset which does not meet the standards-based approach described in FRS can be upgraded to meet modern engineering standards by using a risk-based approach on a section-by-section basis. Adding an emergency spillway and regulating the crest profile has increased the resilience of flooding in the private gardens. The project has also demonstrated how construction challenges associated with a constrained site, working in a public space and dealing with invasive and protected species can be managed successfully to limit the environmental impact and provide the public with an improved open space for recreation.

Cove Brook Flood Storage Reservoir – upgrading to meet modern reservoir safety standards



Figure 6. Emergency spillway embankment upon completion

ACKNOWLEDGEMENTS

The authors would like to thank Stephen Naylor from the Environment Agency and Alan Brown, Jacobs ARPE, for their support in preparing this paper.

REFERENCES

- Atkins (2019), *Cove Brook Flood Storage Area. Report on an inspection under Section 10(3) of the Reservoirs Act.*
- ICE (2015), *Floods and Reservoir Safety. 4th Edition.* Institution of Civil Engineers, London, UK.
- Jacobs (2021), *Cove Brook Flood Storage Area. Assessment of resilience of embankment to overflow.*
- Jacobs (2024), *Design Report (Document No: ENV0004168C-JAC-ZZ-00-DS-Z-0001)*



The British Dam Society

Underwater Enabling Works for a Scour Culvert Gate Overhaul at Leixlip Dam

K VON LOGGERENBERG, ESB
C SMITH, ESB

SYNOPSIS

Leixlip Dam, a 1940s mass concrete structure on the River Liffey, incorporates a scour culvert which provides emergency drawdown capability and forms a critical component of the dam safety system. To complete the scheduled 20-year overhaul of the culvert sluice gate, ESB delivered underwater enabling works to establish an upstream isolation and a dry working environment within the culvert while maintaining reservoir integrity. The enabling solution, informed by industry engagement and risk assessment, adopted a limpet cofferdam. Pre-works diver inspections and 3D scanning confirmed geometry/condition and identified significant silt accretion restricting access. A legacy steel cofferdam frame was refurbished following blast cleaning and NDT, and new D70 hardwood beams were fabricated.

Design enhancements to the historical isolation included neoprene inter beam seals, improved wedging and blocking arrangements for positional stability, and an extended filler valve spindle (to facilitate controlled equalisation and depressurisation). Structural checks confirmed capacity to resist approximately 20m hydraulic head (~95 tonnes resultant load). An initial installation highlighted tolerances and sealing challenges, resulting in targeted design modifications. Following re-installation, leakage under head was controlled using pumped dewatering, allowing safe scour gate removal, its off-site overhaul, and reinstatement of the scour gate over a six-week period. The cofferdam was subsequently depressurised, removed, and retained for future use.

This paper presents the methodology, construction sequence, safety controls, performance monitoring, and key lessons learned for cofferdam-based isolations in ageing dams where forming beam slots or full reservoir drawdown is not practicable.

BACKGROUND AND OVERVIEW OF LIFFEY CATCHMENT:

The River Liffey Hydro-electric Scheme, owned and operated by ESB, comprises three Category A concrete dams downstream of the Pollaphuca Reservoir (*Figure 1*). The scheme serves multiple functions including electricity generation, water supply, recreational use, and flood

Underwater Enabling Works for a Scour Culvert Gate Overhaul at Leixlip Dam

management. This paper relates to the Leixlip dam which is the furthest downstream of the three Category A concrete dams on the scheme. Leixlip Dam occupies a strategically sensitive position due to its proximity to a major potable water treatment plant. Any unplanned drawdown or operational failure would have immediate consequences for Greater Dublin's water supply.

INTRODUCTION

Leixlip Dam is a mass concrete gravity structure, with an operating head of approximately 20m, crest length of approximately 114m and an installed capacity of 4MW. The scour culvert at Leixlip Dam provides a critical emergency drawdown function. A mechanically operated steel sluice gate, located mid-culvert at the base of the dam, retains the upstream water head during normal operations (*Figure 2*).

The scour gate within Leixlip dam was due for its 20-year maintenance overhaul. Unfortunately, no permanent isolation dam beam facility was originally provided during the construction of the dam which meant that temporary works were necessary to safely access the culvert and facilitate gate removal. This required the installation of both upstream and downstream isolations while maintaining reservoir levels and operational continuity. The temporary works used to retain the upstream reservoir head consisted of a steel frame infilled with timber dam beams, creating a limpet cofferdam. This steel frame was an existing bespoke structure that was fabricated previously for the Leixlip scour culvert to create a cofferdam for periodic overhauls. Prior to this overhaul it was understood that this steel frame was last employed in 1998.

This paper will provide a summary of the works that took place to renovate, assemble, install and remove the upstream limpet cofferdam. In addition, the paper will also provide details of the temporary dam beam system which was used to provide an isolation from the tailrace on the downstream side of the scour gate.

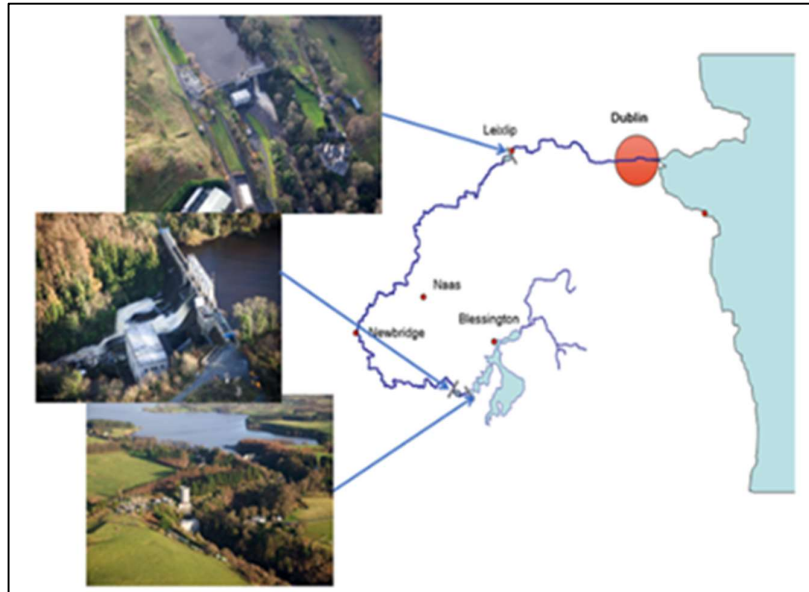


Figure 1: Overview of the Liffey Hydro Scheme.

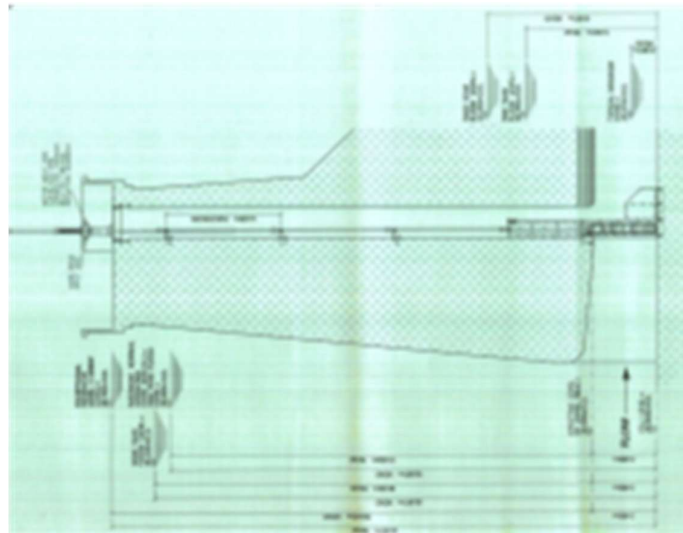


Figure 2: Section Showing Location of the Scour Gate within the Dam Structure

PREWORKS SURVEYING

A comprehensive survey programme was undertaken to inform design and installation methodology:

1. Survey 1 – Underwater Inspection:

Confirmed condition of the intake cage, verified removability of top bars, and identified no significant internal sediment within the cage, see Figure 3 for the sketch provided on the report.

Underwater Enabling Works for a Scour Culvert Gate Overhaul at Leixlip Dam

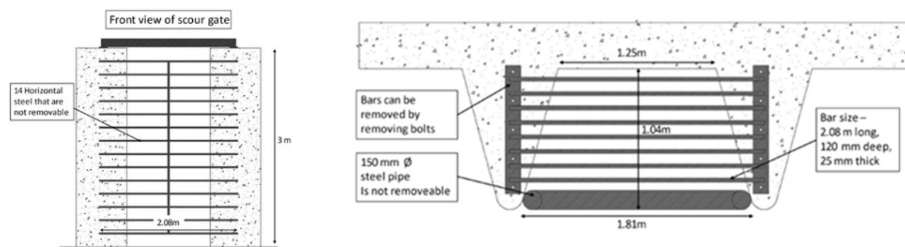


Figure 3: Sketches from Survey 1

2. Survey 2 – Underwater Dimensional and Structural Survey:

Provided detailed geometry of upstream and downstream culvert zones. A 3D laser scan generated point cloud data of the intake structure and surrounding concrete, capturing both internal geometry and external silt profiles (Figure 4). A structural condition survey was also carried out at the same time.

3. Survey 3 – ROV Survey:

This survey was undertaken by a remotely operated vehicle (ROV). This ROV attempted a visual inspection of the outside of the scour culvert cage, but visibility was low and therefore the data was not informative. However, additional point cloud data confirmed significant silt build-up around the intake (Figure 5).

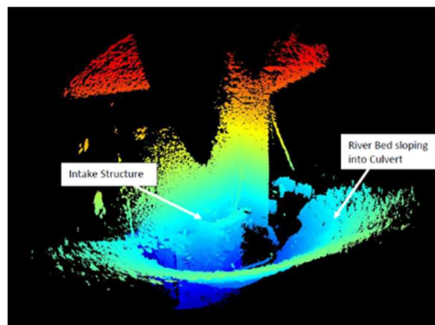


Figure 4: Point Cloud Overview from Survey

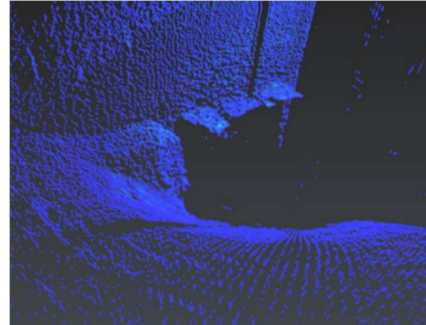


Figure 5: Screenshot of the scanning data.

OPTIONEERING

Following the survey work, a formal optioneering study assessed feasible upstream and downstream isolation solutions, considering constructability, risk, and operational constraints. This exercise was done to review the previously applied methodology for the scour gate vs different options and was considered a crucial step given that the previous overhaul was undertaken more than 20 years previously. The options report presented the

following approaches for each location (upstream and downstream of the scour culvert):

Upstream:

1. A cofferdam installed external to the intake screen.
2. A limpet cofferdam installed internal to the intake screen.
3. A bulkhead installed downstream of the scour gate.

Downstream:

1. A sandbag dam within the mouth of the culvert.
2. A water filled dam within the spilling basin.
3. A cofferdam gate within the mouth of the culvert.

The options report reviewed and discussed each option and presented the advantages and disadvantages of each. The report recommended that the project should continue with the 'limpet cofferdam installed internal to the intake screen' option for the upstream isolation and the 'cofferdam gate within mouth of the culvert' option for the downstream isolation. This approach balanced safety, programme certainty, and minimised intervention at the intake structure. As part of the optioneering exercise a number of other dam operators in USA and Europe were consulted to understand the industry standard for single point isolations during overhauls.

STEEL FRAME RENOVATION

The historical steel frame was recovered from storage and was inspected for re-use, it was found to be generally in good condition with some paint degradation. It was still decided to renovate the frame to ensure its continued lifespan beyond this overhaul. These activities were carried out off site in a workshop environment (by a mechanical contractor), see Figure 6 which shows the cofferdam in their workshop. A summary of the activities included:

- The steel frame was shotblasted to remove the existing paint and any rust on the steel frame.
- NDT testing was undertaken on the steel frame following stripping of the surface coatings.
- New corrosion protection surface coatings were provided to the steel frame.
- A visual survey of the frame was undertaken to determine if the frame had any obvious warping or twisting. No observations were noted during the inspection.
- New packing / shimming plates were fabricated. A variety of shim plate sizes were produced, to ensure tight fit between the steel frame and concrete.

Underwater Enabling Works for a Scour Culvert Gate Overhaul at Leixlip Dam

- An extended spindle was added onto the filler valve to allow operation of the valve from outside of the scour culvert cage during the cofferdam removal process.

TIMBER BEAM VERIFICATION AND MODIFICATION

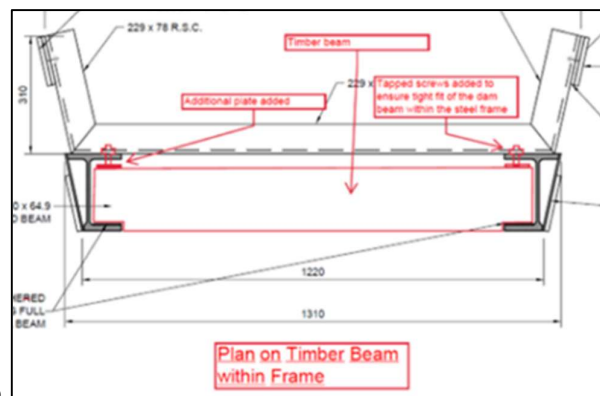
The timber beams used within the cofferdam were greenheart timber which has a strength class of D70 and durability class of 1. The timbers were modified to machine in “U” loops (8mm in diameter) for lifting. Stainless steel plates 50mm x 6mm were also fixed to the ends of the timbers, to assist with sealing and fit. The timbers were inspected by a timber expert who confirmed the timbers strength and condition. Their report confirmed the timbers met the requirements of BS 5756 “Visual Strength Grading of Hardwood”.

COFFERDAM ASSEMBLY

A document for the assembly of the cofferdam was formalised and issued to the mechanical contractor. The cofferdam and timber dam beams were then sent to the mechanical contractor’s workshop for renovation and assembly. Following the renovation of the cofferdam, the timbers were assembled within the steel frame. During assembly it was found that due to dimensional varieties, the timbers did not fit tightly within the steel frame. ESB visited the fabrication workshop to discuss this issue and other items relating the cofferdam construction. Subsequently the items below were instructed to be actioned by the mechanical contractor:

- Plane the timbers to standard sizes (152mm x 305mm).

A second flat plate was added to the other side of the timbers to prevent damage when the timbers are compressed into position. Tapped screws were also installed to ensure the timber dam beams were tight within the



frame, see

- Figure 7.
- The edges of the timbers were sanded to the same radius as the root radius of the steel for a tight fit.

- Timber dam beams were soaked in water for 5 days prior to insertion into the steel cofferdam frame.
- Neoprene seals were added to the frame and secured via epoxy resin.
- Lifting points at the top of the cofferdam were certified for a SWL of 3 tonnes.
- The timber dam beams were secured within the frame with steel packing wedges.

Figure 6: Limpet Cofferdam in Mechanical Contractors Workshop During Renovation

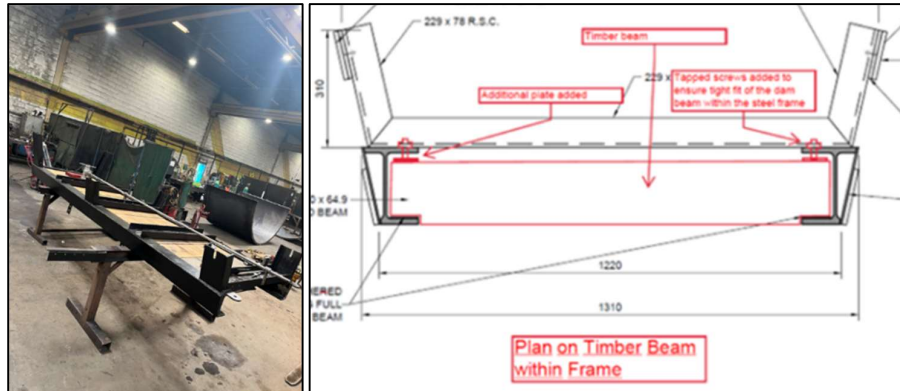


Figure 7: Revision to Timber Beams Within Mechanical Engineering Workshop.

The above items were actioned by the mechanical contractor and a follow-up inspection of the assembled cofferdam was undertaken prior to the first mobilisation. The completed overhauled cofferdam can be seen in *Figure 8*.

ENABLING WORKS – INSTALLATION

Upstream Limpet Cofferdam

The methodology for installation of the cofferdam followed the outlined sequence set out in the particular specification; this methodology was also expanded on in the diving contractor's method statement.

1. Mobilisation to site of all plant, equipment and personnel.
2. Assist with transport of the existing cofferdam to the top of the dam.
3. Crack open scour gate to clean cill under proposed cofferdam location.
4. Prepare the entrance of the scour culvert for receiving the cofferdam e.g. clean any sediment at base, clean vertical concrete walls and remove top of intake screen.
5. Lift existing limpet cofferdam into the water.
6. Divers descend and guide the cofferdam into the intake screen. Divers to remain outside of the intake cage until the blocking pieces are installed back to the steel cage. Divers to take care to stay on the

Underwater Enabling Works for a Scour Culvert Gate Overhaul at Leixlip Dam

upstream side of the cofferdam. Divers to descend into the intake screen with the cofferdam to guide it into position. At no stage should the diver be under the cofferdam while it is in a suspended condition.

7. Diver wedges cofferdam into position, coarse intake screen to be used as aid to assist with wedging.
8. Once the cofferdam is in place, the diver will insert the packing plates to ensure cofferdam fully wedged into position.
9. Divers to move out of the intake screen (to surface) for the initial cofferdam seal testing.
10. Slightly open the scour gate to drain the area between the limpet cofferdam and scour gate.
11. Effectiveness of the seal to be assessed. If seal is not fully effective the divers will use rags, to enhance seal.
12. Diver to carry out final upstream inspection.
13. The flow of water through the scour culvert to be monitored for at least 24 hours to observe functionality of the seal.
14. Once functionality of seal is established the cofferdam should be inspected from the downstream side.

A first attempt to install the cofferdam was undertaken with the diving contractor and crane supplier, see *Figure 9*. This attempt to install the cofferdam reached point 7 in the above sequence when it became clear that there was a gap of 50mm plus between the top seal and the concrete face. The other three sides of the seal between the concrete and the cofferdam were checked, and these seals were found to have a tight fit with the rubber seal in contact with the concrete in all locations.



Figure 8: Renovated Limpet Cofferdam On-site Ready for Installation



Figure 9: Photo of the Limpet Cofferdam being Lifted into Position for the Diving Contractor

It was decided that the cofferdam would be removed from the water and modifications would be made to the top seal to provide a better fit as it was believed that the gap was too large to rag. The modifications made to the top seal included the addition of 2 No. 63mm thick hollow “D” PVC seals to the face of the top timber beam (

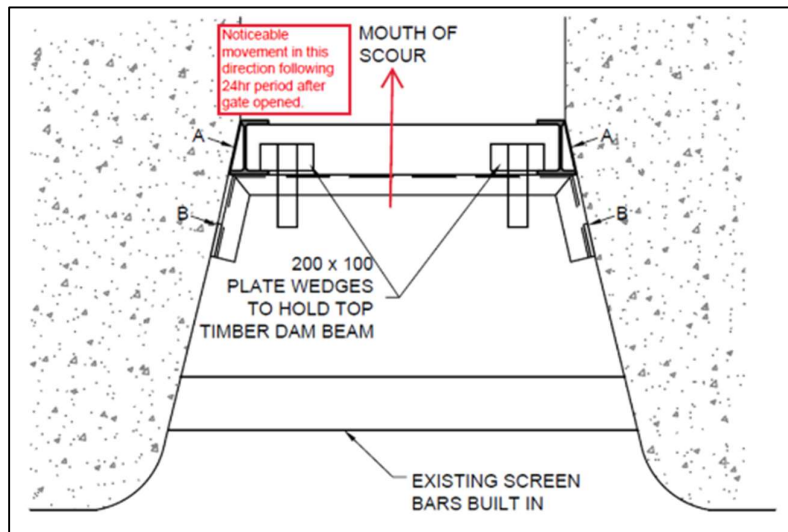


Figure 11: Sketch of the Movement of the Limpet Cofferdam following Opening of Scour gate

Underwater Enabling Works for a Scour Culvert Gate Overhaul at Leixlip Dam

Downstream Cofferdam

The downstream cofferdam was formed using PFC's (modified with a UEA welded to allow for fixings). These PFC's were bolted to the wall using resin fixed stainless steel bolts. The flanges of the PFC's were then infilled with timbers which were sealed against a rubber strip on the inside of the flange of the PFC.

).

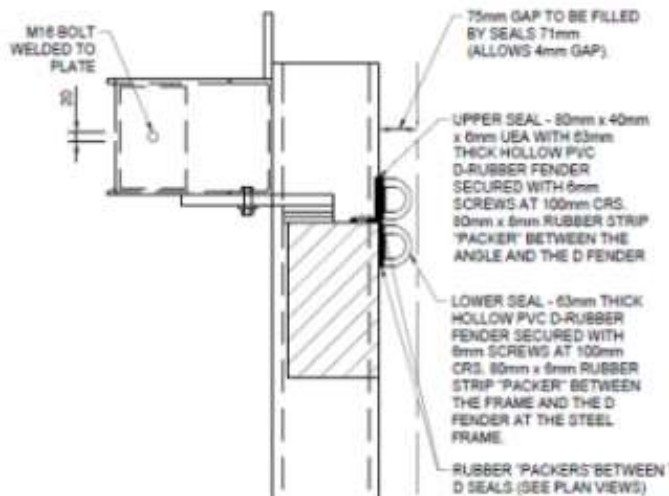


Figure 10: Revised Detail to Top Dam Beam

A second attempt at installing the cofferdam was then attempted, on this occasion the cofferdam was installed with minimal gaps achieved on all sides. Some ragging was required down the left seal, top seal, between the steel frames flange / timbers and between the timber dam beams. At the base of the cofferdam timber wedges were inserted under the bottom seal where the gap was slightly larger.

Once the ragging and wedges were in place the divers exited the water and the scour gate was cracked open. There was an initial surge of water which discharged the existing water between the limpet cofferdam and the scour gate. Following this initial surge there did not appear to be a significant flow from the scour culvert and the scour gate was left open for 24 hours to verify the seal adequacy of the limpet cofferdam.

24 hours later the divers returned to the cofferdam to review the seals. There was a slight turbulence within the scour culvert (observed from the top of the shaft at crest level). The divers found that the cofferdam had moved towards the concrete surface, the pressures on the cofferdam

appeared to have pushed it inwards (*Figure 11*). This movement also resulted in the rags to the top seal being pulled through by the pressures, the top seal had noticeably compressed creating a good seal. The divers proceeded towards the cofferdam with a draft tell-tale searching for any differential pressures ahead of them in order to ensure the area was safe to work within.

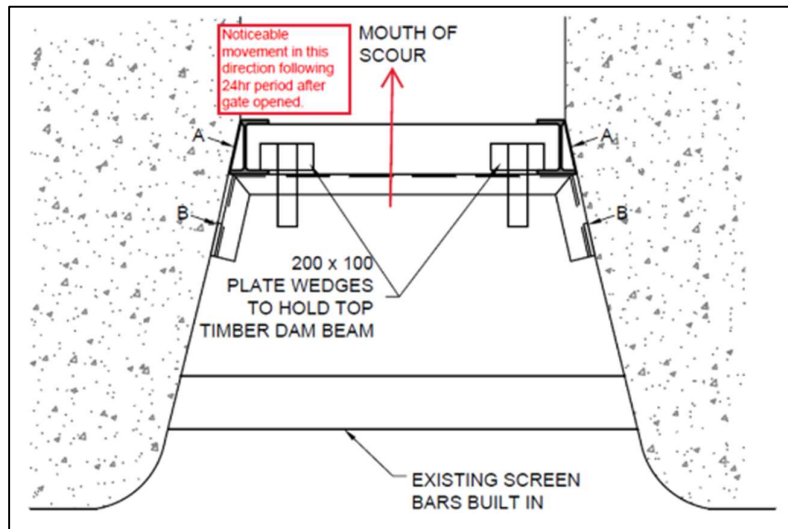
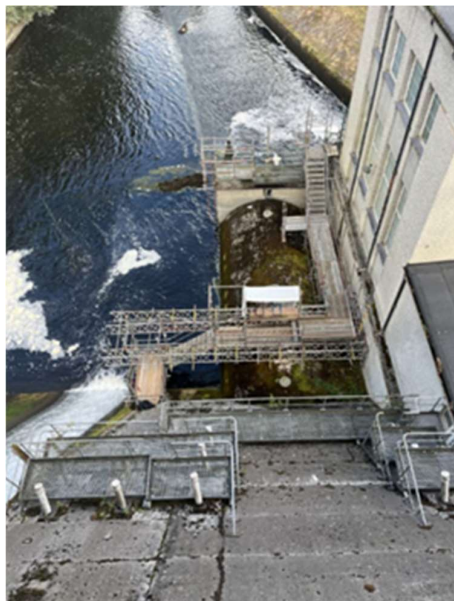


Figure 11: Sketch of the Movement of the Limpet Cofferdam following Opening of Scour gate

Downstream Cofferdam

The downstream cofferdam was formed using PFC's (modified with a UEA welded to allow for fixings). These PFC's were bolted to the wall using resin



Underwater Enabling Works for a Scour Culvert Gate Overhaul at Leixlip Dam

fixed stainless steel bolts. The flanges of the PFC's were then infilled with timbers which were sealed against a rubber strip on the inside of the flange of the PFC.

Figure 12: Location of Scaffold on from Fish Pass to Downstream Isolation

To facilitate access to the scour culvert from the downstream side of the dam a scaffold had to be built from the fish pass (Figure 12). This scaffold was to be used as an emergency stretcher exit from the scour culvert when works to remove and reinstall the scour gate took place. The cantilevered scaffold was fixed to the wall of the power station building (uplift reaction) and rested onto the top of the fish pass structure (downward reaction), then cantilevered out beyond this position by 6680mm to reach the scour culvert.

ENABLING WORKS – OPERATION

Following installation of the downstream cofferdam the scour culvert was pumped out to observe the leakage occurring from the upstream limpet cofferdam. To clear the area 2 No. submersible pumps (capacity of 1800l/min) were required (Figure 14). The pumps were only turned on periodically when access to the culvert was required. Once turned on the effectiveness of the pumps reduced over time due to debris clogging up the intake of the pumps. When the pumps were turned off, the space filled up to the top of the downstream temporary dam in approximately 30 minutes.

A visual inspection of the downstream face of the upstream limpet cofferdam was then undertaken by the station, see Figure 13. There was leakage from between the timbers and between the steel flanges and the timbers. Given that the cofferdam was retaining circa 20m of water head, the seal that was obtained was considered good.



Figure 13: Downstream Face of the Upstream limpet Cofferdam



Figure 14: Installed Downstream Cofferdam with Pumps

Following successful installation of the isolations the mechanical contractor mobilised to site to remove the scour gate and commence its overhaul. When the scour gate was fully opened it jammed in the open position and could not be closed again. A series of props were installed under the scour gate to facilitate its safe removal.

Once the scour gate was removed another inspection of the downstream face of the limpet cofferdam was undertaken to ensure that there were no signs of distortion or stress on the limpet cofferdam system and that leakage was at an acceptable level. This was undertaken as the limpet cofferdam was to act as the scour culvert isolation during the off-site renovation of the scour gate. A measurement staff gauge was installed at the downstream exit of the scour culvert and this was monitored on a continuous basis via SCADA by ESB's Hydro Control Centre with the station attendant visually checking the level of flow from the upstream isolation on a daily basis. Any changes in flow would indicate issues with seal / positioning of the isolation.

SCOUR GATE OVERHAUL

The scour gate was removed by the mechanical contractor and was taken to their workshop and renovated. See *Figure 15* and *Figure 16* which show the scour gate in-situ and removed ready for transportation. The scour gate was then reinstalled and dry commissioned. The gate was then closed and left

Underwater Enabling Works for a Scour Culvert Gate Overhaul at Leixlip Dam

closed for a number of weeks while the diving contractor and crane were scheduled for the removal of the cofferdams.



Figure 15: Scour Gate in Position Prior to Removal



Figure 16: Scour Gate Removed and Ready for Transportation

The method statement from the diving contractor was reviewed and agreed; this outlined that the downstream cofferdam would be removed first before the upstream limpet cofferdam.

ENABLING WORKS – REMOVAL

Downstream Cofferdam

The downstream cofferdam was removed first, as any surge of water from opening the scour gate during the wet commissioning would be allowed to exit without being impeded by the downstream cofferdam. The removal was done by the divers with waders and was accessed from the spillway. The divers gained access to the spillway via a ladder from the fish pass, as the scaffold used for access during the main works had been removed. The wedges, plastic sheeting, sandbags, timbers and steel channels were removed and recovered to the station for storage. The bolts installed into the concrete wall remained, so that they could be used again. Extra nuts were installed onto the exposed thread of the bolts to ensure maximum lifespan.

Upstream Limpet Cofferdam

Removal of the upstream limpet cofferdam required controlled depressurisation via the filler valve on the cofferdam and with the scour gate closed. The divers entered the water and approached the scour gate intake cage. The divers ensured that the blocking pieces were in place, they then entered the screened area and removed the blanking plate from the equaliser valve. The extended spindle was then attached to the valve and

the diver exited the intake cage to operate the valve from the top of the concrete wing walls.

Once the valve was opened the sound of running water could be heard from the scour gate which was unexpected. It was expected that the void between the scour gate and the cofferdam would have been filled by the leakage through the cofferdam since the scour gate was thought to be closed. The valve was left in the open position for 24 hours, the divers then returned to the top of the wing walls at the scour culvert. The sound of flowing water could still be heard and it was evident that there was an issue with the seal of the reinstalled scour gate.

The removal process was stood down for the weekend, the divers and crane provider returned on the following Monday. The station re-checked the scour gate level calibration and found that it was possible to lower the scour gate manually even though the gauge indicated that it was fully shut. This gauge was found to be broken. Once the gate was fully closed manually, the water level within the scour culvert could be seen rising from the top of the shaft at crest level. The divers returned to the upstream limpet cofferdam and the sound of running water was no longer evident. The divers entered the intake screen area using their differential pressure tell-tale at the intake valve and no differential pressures were encountered. The upstream limpet cofferdam could then be safely removed, first removing the shims, then using a hydraulic jack to dislodge the cofferdam from the wing walls and finally lifting the cofferdam to the surface.

CONCLUSION

In conclusion, the Leixlip Dam scour culvert gate overhaul demonstrated that a carefully planned and adaptively managed limpet cofferdam solution can successfully provide a robust upstream isolation, even within constrained and ageing infrastructure lacking dedicated dam beam slots. The combination of detailed pre-works surveying, targeted refurbishment of legacy equipment, and iterative design enhancements were critical to the success of the project.

While leakage was present, it remained manageable through pumping and monitoring, enabling the safe removal, refurbishment, and reinstatement of the scour gate. The project also highlighted the importance of verification of mechanical systems, such as gate position indicators, and the value of contingency planning and real-time problem solving during installation and removal phases. Overall, the works provides a reference for cofferdam-based isolation at operational dams, demonstrating that robust temporary

Underwater Enabling Works for a Scour Culvert Gate Overhaul at Leixlip Dam

works solutions can be achieved without full reservoir drawdown or permanent structural modifications.

ACKNOWLEDGEMENTS

The authors wish to acknowledge the valuable contribution of the specialist contractors who supported the successful delivery of these works. In particular, recognition is given to Norfolk Marine for their expertise in diving operations and underwater installation activities, which were critical to the safe and effective positioning, inspection, and removal of the cofferdam system.

The authors also acknowledge Fenelon Engineering, whose role in the refurbishment, modification, and assembly of the limpet cofferdam structure and timber beam system was instrumental in achieving the required performance under significant hydraulic loading conditions.

The collaborative approach adopted between ESB and the project contractors was a key factor in the successful execution of the works, particularly in addressing unforeseen challenges during installation and removal phases.



The British Dam Society

Rehabilitation of the Underdrainage System at Cow Green Reservoir

A WEBSTER, Stantec

I HUTCHINSON, Stantec

B DOOLEY, Stantec

S LIGHTFOOT, Northumbrian Water Limited

SYNOPSIS

The construction of Cow Green Reservoir near Barnard Castle was completed in 1970 and the reservoir is impounded behind a 25m high, 550m long dam across the valley of the River Tees in Durham. The dam comprises a gravity concrete structure across the central section and left flank, which ties into an earthfill dam embankment on the right flank.

The gravity dam was designed with an internal drainage gallery to intercept seepage and leakage via a series of relief holes and drains. Their purpose is to prevent a build of uplift pressure within the dam and its foundation. The system incorporates 93 relief drains, 90 pressure relief holes, 90 interconnecting carrier drains and approximately 360m of interconnecting pipework.

Calcium carbonate and other salts leached from the concrete have been precipitated within the drainage network over time and had built up to the extent that parts of the system were either partly or wholly blocked, thereby preventing the system from functioning as originally designed. In 2023, the Inspecting Engineer made several recommendations in the interests of safety, one of which included the removal of mineral deposits from the drainage system, while another sought to improve the leakage measuring system.

Previous attempts to clean the system had been unsuccessful due to poor access for cleaning, the hardness of the deposits, not to mention the limitations associated with confined space working. The drainage measurement system was uncalibrated and in need of modernisation. This paper outlines the works undertaken to inspect and clear the underdrainage system along with capital works to install telemetry, thereby increasing the reliability and reducing the demands on operations personnel.

INTRODUCTION

Cow Green Reservoir is a large composite dam across the River Tees, about 15km northwest of Middleton-in-Teesdale in the County of Durham. The

Rehabilitation of the Underdrainage System at Cow Green Reservoir

dam comprises of a concrete gravity element in the central section and left flank and an earthfill embankment on the right flank.

A drainage gallery is situated within the concrete dam structure, where a series of relief wells and drains allow the seepage to be relieved and drained, which then flows through a carrier pipe and is measured in the instrument room. The collective drainage system prevents build-up of water pressure within the dam, contributing to the stability of the structure and maintaining reservoir safety. Changes in the observed flows over time could provide indications of potential problems developing within the internal structure of the dam and so it is important that the system is functioning as intended.

In May 2023, a statutory inspection of Cow Green Reservoir was conducted under Section 10(3) of the Reservoirs Act 1975. The associated report specified several measures to be undertaken in the interests of safety, including the removal of calcium carbonate deposits that constrain the pressure relief flow along with enhancements to the leakage-measurement system.

A period of on-site rehabilitation works were undertaken over a ten-week period from November 2025 to January 2026. Stantec was appointed as Principal Designer, Esh appointed as the Principal Contractor, with the confined spaces access and physical works subcontracted to Triton Risk Management.

Originally anticipated to comprise a trial period of works due to the numerous constraints faced, the development of an efficient cleaning methodology enabled for the successful rehabilitation of the underdrainage system which delivered significant efficiencies in both time and budget. The same methodology is now being applied to other reservoir sites in need of routine modernisation.

PRESSURE RELIEF DRAINAGE BACKGROUND

The pressure relief drainage infrastructure, located within the drainage gallery of the concrete dam, is approximately 23m below the dam crest at its deepest point at the base of the valve tower. Divided into eastern and western galleries either side of the central valve shaft, the eastern gallery measures approximately 200m in length, with the western gallery being approximately 45m long.

The galleries are narrow confined spaces, approximately 1.2m wide and 2m high. As shown in Figure 1, access to the drainage gallery is via a staircase located within the valve tower, which is accessed from the dam crest.

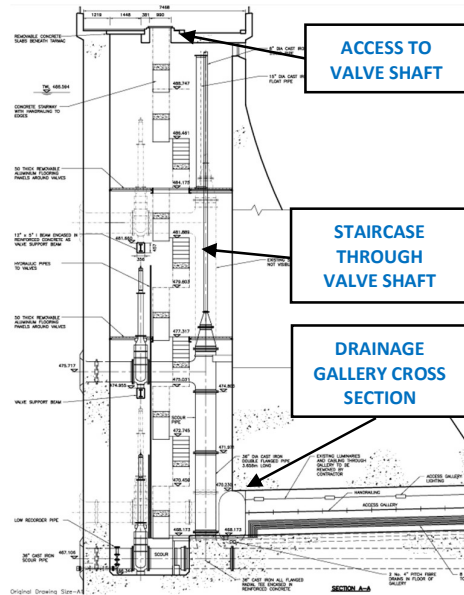


Figure 1. Access to the drainage gallery through valve shaft staircase



Figure 2. Photograph taken within the eastern drainage gallery

Existing Drainage Infrastructure

The drainage system comprises a series of interconnected pipework which collectively intercept seepage and uplift flows within the structure. This includes:

- 90 pressure relief holes formed within the concrete structure aligned across the axis of the dam, located at the bottom within a recess in the gallery wall on the upstream side. The pressure relief holes are indicated in yellow on Figure 3.
- 93 pressure relief drains extending down to bedrock. The pressure relief drains are located beneath gratings located on the gallery floor. The relief drains are indicated in red / brown on Figure 3.
- A 102mm pipe captures flow from the pressure relief holes and combines with the flow from the relief drains. This is shown in blue on Figure 3.
- A 152mm carrier pipe captures all drainage from the pressure relief holes and relief drains, spanning the length of the galleries. This passes to a single pipe that carries flow from the drainage gallery to the instrument room where the flows are measured. The carrier pipe indicated in purple on Figure 3.

Rehabilitation of the Underdrainage System at Cow Green Reservoir

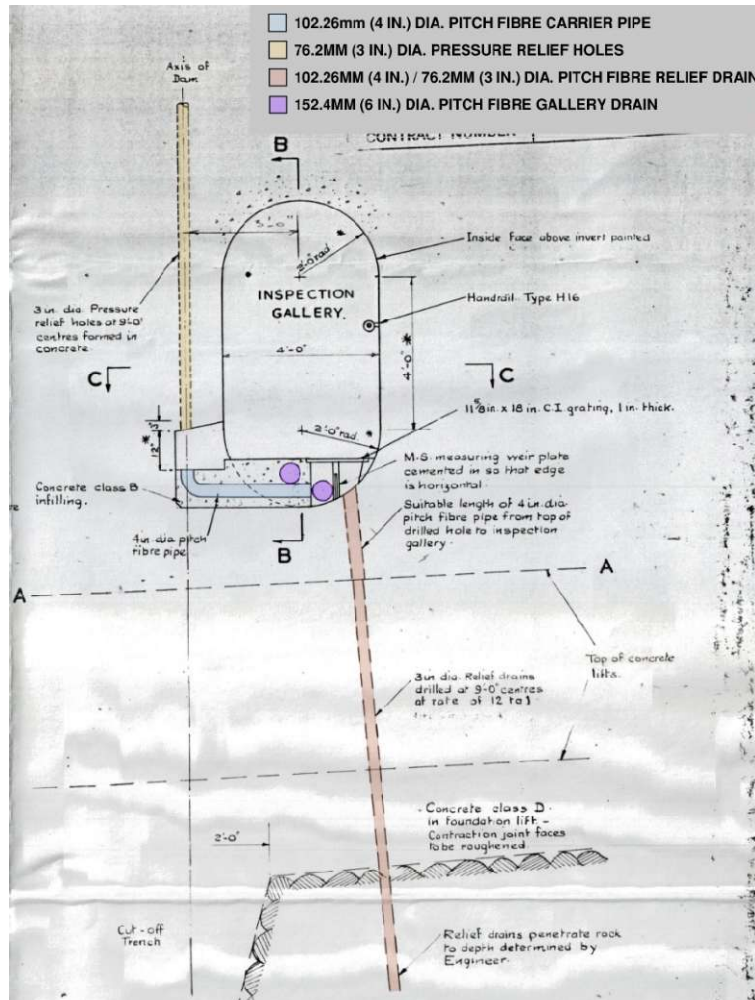


Figure 3. Extract of a record drawing with highlighted drainage and corresponding legend

REQUIREMENT FOR CLEANING WORKS

Within the pressure relief system, calcium carbonate and other salts naturally leached from the concrete have gradually accumulated over many decades. Due to the physical access and confined space constraints preventing effective cleaning through maintenance, the mineral deposits had accumulated to a level in which parts of the system had become partially or fully obstructed, reducing system functionality. From Figure 4 below, solidified mineral deposits can be seen extending beneath the inlet to the vertical pressure relief hole (accessed on the soffit of the concrete cut-out section). In addition, the inlet to the carrier pipe was not visible at all.

In instances where flow was observed from the pressure relief holes, it was often spilling onto the gallery floor rather being captured by the carrier pipe. On the right, Figure 5 shows the same pressure relief hole following cleaning. All solidified calcium carbonate deposits were removed and the inlet to the carrier drain is now visible. Following the removal of blockages from the pressure relief holes, the sub-contractor noted that frequently an immediate release of water followed, demonstrating it was necessary to free the system from deposits.



Figure 4. An example of a blocked pressure relief hole prior to cleaning works.

Figure 5. An example of a pressure relief hole after cleaning.

DEVELOPMENT OF AN EFFICIENT REFERENCING SYSTEM

To assist in discharging a different measure in the interests of safety, an exercise was undertaken to map the existing drainage present within the drainage gallery. A series of detailed point cloud laser scans were taken through the space, which allowed for the development of a comprehensive drawings package, mapping out the location and details of the infrastructure within.

The drawings package developed provides a unique label for each pressure relief hole and relief drain within the gallery, and a drawing sheet for each individual monolith that the gallery traverses through.

For coherence and efficiencies, it was agreed that the referencing system developed should be used for all other associated works within the space.

Rehabilitation of the Underdrainage System at Cow Green Reservoir

Considering this, prior to any works being undertaken within the gallery, an exercise was carried out to label each pressure relief hole, relief drain and monolith joint to allow for effective record keeping during the works. Permanent tags are to be installed within the gallery to ensure the referencing system is maintained and to support ongoing monitoring and maintenance.

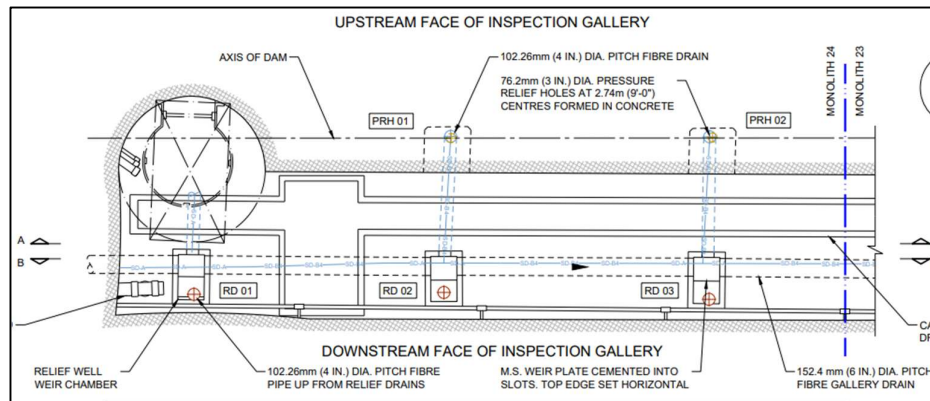


Figure 6. Extract of the plan view of an individual monolith drawing mapping the drainage infrastructure

DEVELOPMENT OF THE CLEANING METHODOLOGY

A key challenge presented by the works was the absence of recorded case histories of similar schemes which have undertaken the rehabilitation of a pressure relief drainage system of this configuration. Due to this, there was significant uncertainty regarding the effectiveness of proposed cleaning techniques. As such, an initial trial period was proposed to understand the effectiveness of various methods, with plans to revisit site with more intrusive methods if required. This could have entailed removal of concrete surrounding the well collars of the pressure relief holes, to improve access.

The initial rehabilitation methods identified the use of high-pressure water jetting combined with conventional mechanical ferrets to remove the mineral deposits. The use of this method during the initial stages of the on-site works proved largely ineffective due to the hardness and extent of the calcium carbonate deposits, which in many instances exhibited a rock-like hardness.

Difficulties also arose due to the geometry of the pressure relief holes and the limited space available within the access pockets in the walls. The geometry of the drainage infrastructure restricted the use of larger equipment. In addition, the complicated access arrangements to bring heavy equipment into the space combined with the risks of using heavy machinery in a confined space further ruled out the use of heavy machinery.

To overcome these challenges, a bespoke mechanical cleaning tool was developed on-site. The system utilised a flexible wire cable fitted with a conical drill attachment powered by a cordless high-torque drill. This new arrangement provided an easily manoeuvrable device and allowed the operators to effectively break up the hardened deposits within the pressure relief holes and connecting pipework. Following the use of the cleaning tools, residual deposits within the system were removed through flushing, leaving a freely flowing system free from deposits. Figure 7 below shows the tool used to undertake the cleaning works.

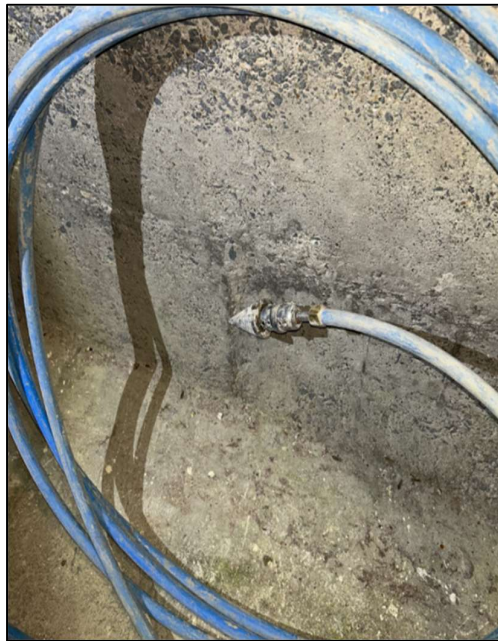


Figure 7. Image of the bespoke mechanical drill attachment used to clean the hardened deposits

RESULTS OF THE REHABILITATION WORKS

With a successful method to remove blockages identified, the blockages were removed in a systematic and effective manner allowing the system to function as originally designed. For verification of the cleaning exercise and to understand the effect of removing blockages, both pre-cleaning and post-cleaning flow surveys were undertaken of the drainage system, including photographs of each asset before and after cleaning. All photographs and flow values were tabulated allowing for a comparison to be undertaken with the observed flows before and after the blockages were removed. In addition, a CCTV survey of the vertical pressure relief holes was undertaken to verify that blockages had been removed.

Rehabilitation of the Underdrainage System at Cow Green Reservoir

Analysis of the post cleaning flow surveys indicates an increase in the standing water levels of the relief drains, potentially indicating that historic blockages have been removed across these drains, allowing the system to operate as designed. An example of this can be seen in Figure 8 below.



Figure 8. Relief drain No.40 prior to cleaning (left) and post cleaning (right)

For the pressure relief holes, the general trend shows that flows increased following the completion of the cleaning works, again indicating that the system is now operating as intended. However, it should also be noted that the reservoir level was slightly higher (1.4m) on the day of the post-cleaning flows survey than the pre-cleaning flow survey, which would have influenced the volume of flow observed through the pressure relief holes. Generally, the flows observed through this system were very low, often being no more than a few drips every 1-2 seconds.

WASTE MANAGEMENT / REMOVAL

Strict environmental screening measures were implemented during the cleaning operations to capture debris and prevent contamination of the downstream watercourse. Using dry vacs and subsequently jetting, all waste materials were collected and appropriately disposed of off-site, ensuring no pollution entered the River Tees. Figures 9 and 10 below show an example of the waste generated during the cleaning works.



Figure 9. Example of solids removed from the drainage system



Figure 10. Example of hardened mineral deposits removed during the cleaning exercise

IMPROVEMENT OF THE UNDERDRAINAGE MEASURING SYSTEM

The flows from all drainage located within the gallery are captured and carried to a manhole located within the piezometer room of the valve house. Two additional manholes are also present within the room in which accumulates the embankment and toe underdrainage for the earthfill dam. Within each manhole is a v-notch arrangement with an accompanying measuring device, which was a punched tape sprocket system that was installed during the original construction of the dam. The system had not recently been calibrated and was approaching the end of its asset life, warranting modernisation. To address this, a new system was installed to accurately measure and record the underdrainage.

New Underdrainage Measuring System

All three manholes within the piezometer room are designated as confined spaces and hence require the use of fall arrest systems and winches to access. Due to this constraint, the solution developed aimed to minimise the requirement for confined space entry into the chambers both during construction and routine maintenance.

The solution comprises a series of four ultrasonic transducers across the three chambers; each attached via a chain and floor mounted hanging plate and junction box. Mounting the transducers in this manner means that for any future maintenance requirements, the transducers can be removed via

Rehabilitation of the Underdrainage System at Cow Green Reservoir

the chain, eliminating the need to enter the confined space. Accompanying the transducers are a series of wall mounted transmitters, calculating the flow. In addition to presenting and storing the readings on the wall mounted transmitters, a new telemetry outstation was also installed under the scheme meaning that readings are automatically logged and fed into the NWG data repository.

To reuse materials where possible and reduce carbon consumption, the original v-notch weirs were retained following adequate cleaning and surface preparation. This reduced the need for the fabrication of new v-notch weirs and bracketry in turn reducing the site waste and carbon footprint.

An example of the old system in comparison to the new system installed can be seen in the figures below.

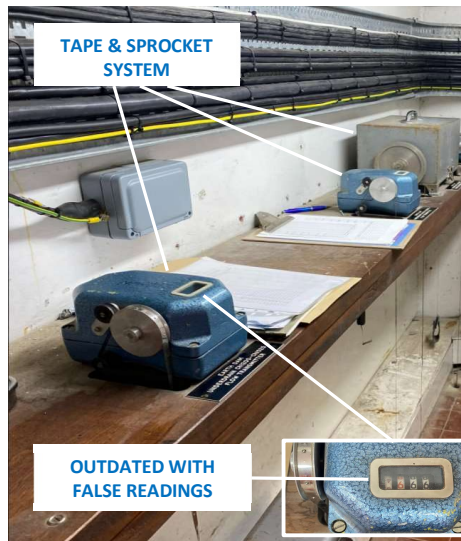


Figure 11. The original flow monitoring arrangement

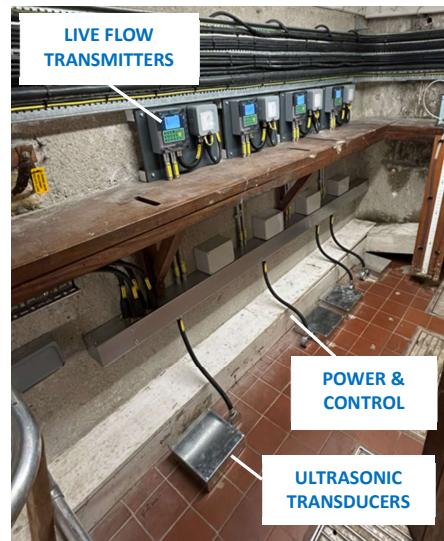


Figure 12. The upgraded flow measuring system

ADDITIONAL CONSTRAINTS

Many constraints were effectively managed during the capital works undertaken at Cow Green Reservoir. The following is a non-exhaustive list of key constraints and how they were effectively managed.

Weather / Remote Nature of the Site

Due to the necessity to meet compliance deadlines, much of the work discussed in this paper was undertaken during the Winter of 2025. Due to the reservoirs high altitude above sea level for the UK, extreme weather conditions are experienced frequently. For two days of the project, work was

paused on-site due to poor accessibility with significant snowfall and ice coverage. In managing this, work was only restarted once temperatures had increased and it was deemed safe to access the site.

Due to the remote nature of the site, there is no phone service available within one mile of the dam crest. This necessitated the installation of a small welfare compound on-site with a Starlink satellite communications system to maintain communication for safety.

Confined Space Constraints

Due to the layout of the drainage gallery, it is classified as a 'medium-risk confined space'. Without appropriate precautions and safe management, confined spaces can present life-threatening risks within a very short period. To address this, it was mandatory for all entrants into the space to hold 'Medium Risk Confined Space' training, with a dedicated rescue team managing person entry, as well as undertaking continuous gas monitoring, with emergency breathing apparatus present if required.

In addition, the confined space nature of the drainage gallery also narrowed down practicable solutions. While methods such as acid etching were ruled out due the risk of pollution to the downstream watercourse, they were further ruled out due to the risk of using hazardous substances in a confined space.

Structural Arrangement of the Pressure Relief Infrastructure

The structural arrangement of the pressure relief holes meant that conventional cleaning methods could not be used and is a large factor as to why the deposits accumulated to such a degree, with all previous cleaning methods proving ineffective. In managing this, flexible tools were developed which meant the pressure relief hole inlets could be accessed, along with working around the 90° bends of the carrier drains.

To monitor the effectiveness of the solutions, strong collaboration was required between the Client, Designer and Contractor to understand and monitor the progress in completion of the works. 'Check-in' points were built in to the programme to review if chosen solutions were making progress, along with frequent site visits allowing for alternative methods to be discussed to address hardened deposits.

CONCLUSIONS

In conclusion, this paper presented the development and delivery of a solution for the rehabilitation of the underdrainage system at Cow Green Reservoir. With the absence of comparable projects, a flexible and iterative approach was adopted, with a clearly defined referencing system used from

Rehabilitation of the Underdrainage System at Cow Green Reservoir

the outset. This allowed for the development and trialling of viable solutions, leading to the implementation of effective methods for the removal of hardened calcium carbonate deposits that were affecting the performance of the system.

In addition, the leakage measurement system was upgraded to align with current industry standards, improving the accuracy and reducing the demands on operational personnel. Overall, the works undertaken have restored the functionality of a dam-safety critical system and extended the service life of the asset. The modernisation of the system ultimately ensures that the reservoir can continue its key role in water supply across the North East.

ACKNOWLEDGEMENTS

The authors acknowledge Northumbrian Water Limited for granting permission to publish this paper and for their continued support throughout the delivery of the project.

The authors would also like to thank the contractor, Esh and the specialist subcontractor, Triton Risk Management for their collaborative approach and significant contribution to the development and implementation of the rehabilitation methodology described in this paper.

The authors further acknowledge the valuable guidance and support provided by Ian Carter, the All Reservoirs Panel Engineer for Cow Green Reservoir, throughout the development and delivery of the works.



The British Dam Society

Capturing and assessing the conditions in concrete dam drainage galleries using novel camera technology and AI image processing techniques

R A COOMBS, CC Informatics

C RAMSAY, SSE Renewables

J CRAMMAN, CC Informatics

M CRAMMAN, CC Informatics

SYNOPSIS

In 2025 a specialist photographic survey was conducted as part of a pilot assessment at a large concrete gravity dam in Scotland. The purpose of the pilot assessment was to determine if recent advances in camera technology were well suited for the collection of data in confined linear spaces such as concrete gravity dam galleries. A site was selected and the survey undertaken in Summer of 2025, and data processing trials were undertaken through the Winter of 2025 and into the early part of 2026.

This paper details the challenges of data collection, addresses the challenges of working with novel camera technology particularly in the context of photogrammetry and 3D model construction, documents alternative means of displaying the photometric information collected in 2D and 3D formats, and demonstrates the potential value in using AI defect detection and tracking methods when considering the structural health of concrete gravity dams – particularly in light of the prior art and other historical records.

INTRODUCTION

Mullardoch Dam

Mullardoch Dam is a large concrete gravity dam that impounds Loch Mullardoch, which forms the main reservoir in the Glen Affric hydro-electric scheme. The dam was commissioned for the North of Scotland Hydro-Electric Board (NOSHEB) in 1951. The reservoir supplies water to Fasnakyle Power Station via a transfer tunnel to Benevean Reservoir and subsequently through a power tunnel to the station. The dam is unusual in plan due to its dual-axis arrangement, a design selected to utilise the site's terrain and reduce construction material quantities. The dam is 727m long and 37.5m at maximum height. The reservoir's capacity is 223Mm³.

Capturing and assessing the conditions in concrete dam drainage galleries using novel camera technology and AI image processing techniques

The dam incorporates an inspection gallery which runs the majority of its length. It serves a dual purpose: monitoring the condition of the dam and housing the pressure relief system. A plan and elevation is given in Figure 1.

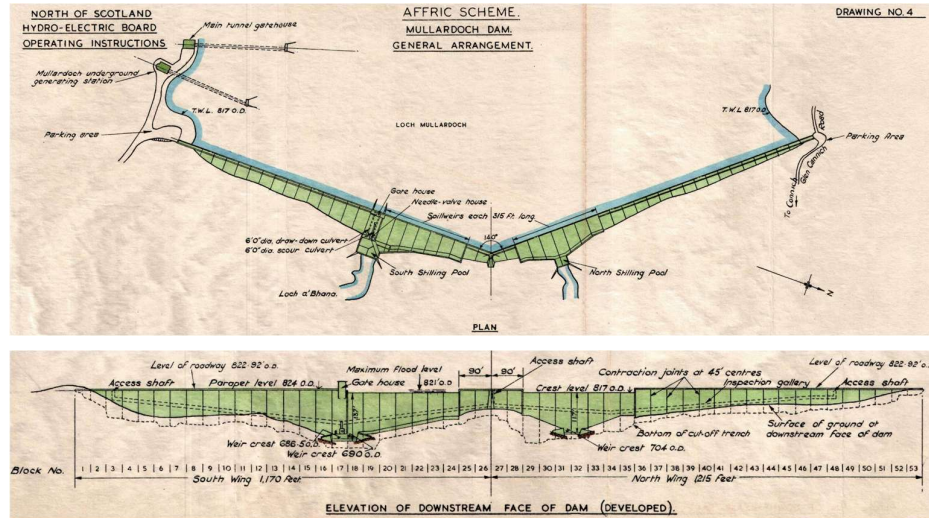


Figure 1. Mullardoch Dam Plan and Elevation

Cracking has historically been identified within the gallery, particularly in areas where concerns existed regarding stress concentration. Symptoms were monitored by means of movement survey data, and evaluation of leakage and cracking in the gallery. Strengthening works were carried out during 1989 and 1990 to prevent further movement by installing 26 No. post-tensioned near-vertical anchors in the central block (Gosschalk, et al., 1991).

Previous attempts to quantify cracking were undertaken in the 1980s through manual mapping of the gallery (see Figure 2). This long process involved carrying out visual surveys which were then transcribed onto proforma drawing sheets. Additional monitoring methods included the installation of tell-tales and, more recently, the use of spray-painted reference markings.

Since completion of the anchoring works and cessation of the associated stress build-up, the condition of the gallery has been monitored through visual inspections rather than detailed crack-mapping surveys. These visual inspections have principally concentrated on identifying areas of significant change such as development of new defects and progression of existing cracking. Little change has been reported during these inspections.

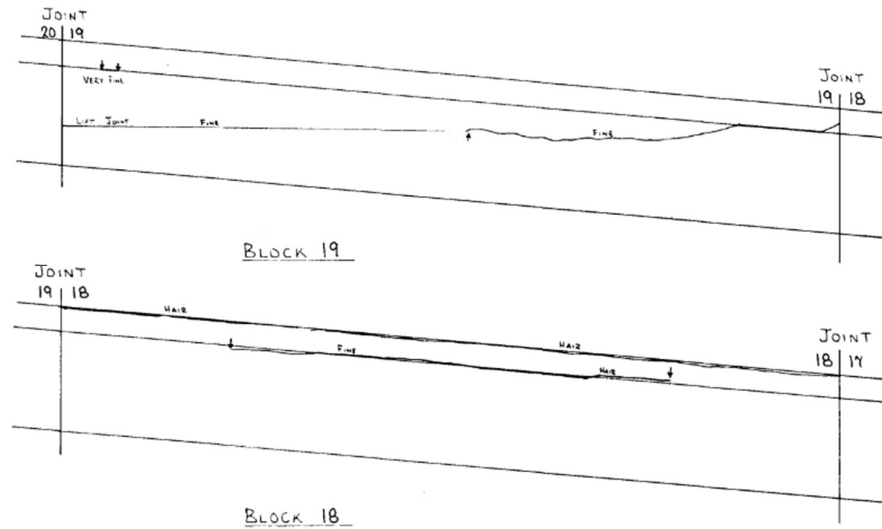


Figure 2. Example of historic crack record drawings, 1989 (Block 18 and 19)

Adoption of an alternative approach

In 2023, the dam was inspected under Section 47 of the Reservoirs (Scotland) Act 2011. Within the report it was recommended that a survey should be undertaken to establish a baseline record and then monitor for any changes in the future (Hewitt, 2023).

The direction under Section 51 (3) of the Act is set out below:

“Access (inspection) gallery: Carry out a detailed survey to record and map crack locations, patterns and their widths. Initial survey to be made within 2 years of the date of this report, with a check survey just prior to the next statutory inspection, or if change in the current cracking is suspected or observed”

Undertaking this survey using traditional manual methods would require significant effort and would be prone to error due to the size of the structure and the number of defects requiring identification and measurement. As such, alternative methods of crack monitoring were investigated for application at Mullardoch. A holistic photographic data collection method was identified as a suitable alternative to manual surveys, particularly owing to recent improvements in camera technology.

Additional benefits of this method include the potential for a reduction in the time required for site data collection and subsequent analysis. The method also addressed several practical challenges, including poor lighting conditions that can make defects difficult to identify and track, the confined nature of the gallery, and the working-at-height risks associated with the access shafts. The creation of a complete digital record of the areas of

Capturing and assessing the conditions in concrete dam drainage galleries using novel camera technology and AI image processing techniques

interest within the structure reduced prolonged personnel exposure whilst providing a detailed and repeatable baseline for future comparison.

To support delivery of the Section 51 direction, SSE Renewables engaged CC Informatics (CCI) to undertake a pilot study investigating whether modern photographic survey techniques could provide a practical and repeatable means of recording the condition of the galleries and shafts. The pilot built upon previous work undertaken by SSE and CCI at another hydro-electric asset, where AI-assisted defect identification and change detection techniques had been trialled using photographic surveys collected approximately one year apart by an external contractor.

The earlier trial demonstrated the potential value of AI as a screening and prioritisation tool rather than a replacement for engineering judgement. Automated post-processing reduced approximately 1,800 identified defects to around 50 locations requiring visual assessment for potential change. Subsequent engineering review rapidly discounted all but one of these locations, with the remaining feature ultimately found to be a cobweb that had been incorrectly classified as a potential crack. This provided confidence that a similar approach could be applied at Mullardoch, where comprehensive manual review of the collected data would be challenging.

DATA COLLECTION

The collection of photographic data from within the galleries and shafts presented several challenges. The galleries are approximately 900mm wide and extend for more than 600m through the structure. Conventional photographic survey techniques typically rely on maintaining sufficient stand-off distance between the camera and the target surface to minimise perspective distortion and provide uniform lighting; however, the confined geometry of the galleries made this impractical. Furthermore, the galleries contain numerous cracks, areas of calcite deposition and staining features, requiring imagery of sufficient quality to permit detailed review following the survey.

Access arrangements within the dam introduced additional constraints. The galleries are reached through narrow corridors incorporating several abrupt changes in direction, making transportation of survey equipment difficult. The gallery floor was also found to be uneven, presenting challenges for the collection of repeatable photographic data over long distances. At either end of the galleries, access shafts provide routes between the gallery and crest level, requiring data to be collected in a vertical orientation whilst maintaining consistent image quality and lighting conditions.

CCI had previously investigated the use of 360° camera systems for the inspection of confined spaces within the UK water and hydro-electric sectors. The ultra-wide field of view allows complete photographic coverage to be collected from short stand-off distances, whilst continuous image capture provides a repeatable dataset suitable for subsequent photogrammetric processing. The Mullardoch pilot study therefore provided an opportunity to assess whether these approaches could be successfully applied to the long, narrow galleries and shafts found within a concrete gravity dam.

Adopted Solutions

To address the access and data collection challenges identified, bespoke survey platforms were developed for both the galleries and shafts. The primary objective of the platforms was to collect repeatable, high-resolution photographic data whilst maintaining consistent camera positioning and lighting conditions throughout the survey.

Gallery Survey Vehicle

The galleries were surveyed using a purpose-built trolley system developed specifically for operation within confined linear spaces (Figure 3). The vehicle was constructed using lightweight modular components to allow transportation through the restricted access routes leading to the galleries and subsequent assembly within the structure. The platform incorporated multiple 360° camera systems and integrated LED lighting to provide complete coverage during a single survey pass.

A particular challenge was maintaining a consistent position within the gallery. To assist the survey operator, a bespoke heads-up display (HUD) was developed using an Arduino-based microcontroller and laser distance sensors. The system provided real-time information on the vehicle's position relative to the gallery walls, allowing centring and alignment to be maintained throughout the survey. This improved the consistency of data collection and supported the repeatability required for future change detection surveys.

The vehicle was manually propelled through the galleries at a controlled speed. Local variations in floor level and construction tolerances required the survey to be undertaken more slowly than originally planned; however, complete photographic coverage of both the upstream and downstream galleries was successfully achieved.

Capturing and assessing the conditions in concrete dam drainage galleries using novel camera technology and AI image processing techniques



Figure 3. Gallery Survey Vehicle

Shaft Survey Vehicle

The access shafts were surveyed using a separate deployment system based upon the same camera and lighting arrangement (Figure 4). The adopted solution consisted of a man access tripod with pulley positioned over each shaft opening, allowing the camera platform to be lowered vertically through the structure in a controlled manner via a slow winch.

The shaft surveys required a stable vertical deployment to minimise rotation and maintain image quality. The camera platform was therefore lowered at a controlled speed to provide sufficient overlap between successive frames and ensure complete photographic coverage of the shaft walls. The integrated lighting system provided consistent illumination throughout the shaft depth, avoiding many of the exposure challenges typically encountered when collecting imagery in deep confined spaces. The pulley system was designed to minimise rotation of the platform on descent and ascent.



Figure 4. Shaft survey vehicle

Repeatability Considerations

A key requirement of the pilot study was the ability to repeat the survey at future dates and compare the resulting datasets. Consequently, the survey methodology was developed around the principles of photogrammetric reconstruction rather than isolated photography. Continuous image capture, consistent lighting conditions and controlled movement of the survey platforms were all intended to maximise the likelihood that future datasets could be processed using the same workflow and compared against the baseline survey.

Whilst the pilot principally sought to establish the feasibility of data collection within the galleries and shafts, consideration was also given to future automated change detection. The repeatability of the photographic survey therefore formed a fundamental design requirement of both survey platforms.

PHOTOGRAMMETRIC DATA MANIPULATION

Photogrammetry

The photographic data collected within the galleries and shafts was processed using Pix4D Mapper to generate photogrammetric models of the surveyed surfaces. The primary purpose of these models was to provide a framework from which photographic information could be accurately

Capturing and assessing the conditions in concrete dam drainage galleries using novel camera technology and AI image processing techniques

aligned and transformed into engineering outputs suitable for review and comparison.

The gallery surveys made use of imagery derived from multiple 360° camera systems mounted on the survey vehicle. Whilst 360° imaging provides significant advantages for data collection in confined spaces, it also presents challenges for photogrammetric processing. The highly distorted nature of fisheye imagery, the proximity of the cameras to the target surfaces and the relatively uniform appearance of long concrete galleries all increase the complexity of model generation. Additional difficulties were encountered where lighting conditions varied or where image quality was affected by movement of the survey platform over uneven surfaces.

Despite these challenges, photogrammetric models were successfully generated for both the galleries and access shafts. The resulting models provided complete coverage of the surveyed areas and formed the basis for subsequent image processing and defect mapping activities.

3D Object Manipulation

While the photogrammetric models provided complete coverage of the galleries and shafts, they were not considered the most practical format for routine engineering review. Consequently, additional processing was undertaken to convert the three-dimensional models into two-dimensional outputs that could be readily viewed, annotated and compared against historical records.

For the main galleries, the photogrammetric models were used to generate high-resolution RGB orthomosaics of the upstream and downstream walls at a target resolution of approximately 1mm/pixel. These orthomosaics combined thousands of individual images into a continuous photographic record of the gallery surfaces, allowing defects to be reviewed within their wider structural context whilst retaining sufficient detail for close inspection.

The access shafts presented a different challenge due to their cylindrical geometry. Conventional orthophoto techniques are well suited to relatively planar surfaces but are less effective when applied to circular structures. To overcome this, the shaft meshes were digitally "unrolled" to transform the cylindrical surfaces into flattened two-dimensional representations. This process allowed the shaft photography to be reviewed in a format similar to the gallery orthomosaics while preserving the spatial relationship between observed defects.

The flattened RGB outputs formed the primary engineering deliverable from the survey and provided a modern equivalent of the historical crack-

mapping drawings. Unlike the photogrammetric models, which required specialist software to interrogate, the orthomosaics and unrolled shaft images could be incorporated directly into PDF drawings and distributed for engineering review. This approach maintained direct traceability between identified defects and the underlying photographic evidence whilst establishing a detailed baseline dataset suitable for future comparison and change detection.

DEFECT IDENTIFICATION AND CHANGE DETECTION

AI-Assisted Review

The survey generated a substantial quantity of photographic data, comprising hundreds of gigabytes of imagery collected from the galleries and shafts. Whilst the orthomosaics and flattened shaft outputs provided a complete visual record of the structure, manually reviewing every image for defects would require a significant amount of engineering effort. An automated method of identifying potential defects was therefore investigated to assist with interpretation of the dataset.

CCI's AssetScan platform was adopted for this purpose. AssetScan is a computer vision system developed to identify and classify engineering defects within photographic datasets. For the Mullardoch pilot study, the system was configured to identify three classes of feature considered relevant to future condition monitoring: cracking, calcite deposition and staining associated with seepage. These features were selected as they provide potential indicators of structural movement, water ingress and changes in leakage behaviour.

The use of this machine vision product formed part of a broader engineering workflow and was not intended to replace engineering judgement. Instead, the AI was used as a screening and prioritisation tool, highlighting areas of interest within the photographic record that could then be reviewed against the underlying imagery. This approach ensured that all identified features remained directly traceable to the source photography whilst reducing the amount of data requiring detailed manual review.

Defect Identification

The AI-assisted review generally proved effective at identifying cracking, calcite deposition and seepage-related staining within the orthomosaics. These features were selected because they provide potential indicators of structural movement, water ingress and changes in leakage behaviour. The approach was particularly beneficial when reviewing the large photographic dataset generated by the survey, allowing engineering attention to be

Capturing and assessing the conditions in concrete dam drainage galleries using novel camera technology and AI image processing techniques

directed towards specific areas of interest rather than requiring exhaustive manual inspection of every image.

As anticipated, the automated process also generated false positives. Examples included construction features such as lift joints and formwork ridges being identified as cracks, electrical cables being classified as linear defects, and areas of overexposed concrete being interpreted as calcite deposits. In isolated cases, non-structural features such as timber instrument housings or localised debris were incorrectly classified as staining. These results highlight the importance of maintaining direct access to the underlying photography and ensuring that all AI-generated outputs remain subject to engineering review.

While the system was not configured to directly measure crack widths, the mapped geometry of identified defects provides a repeatable baseline against which future surveys can be compared. Changes in crack extent, continuity, associated calcite deposition and staining patterns can all be assessed through comparison of successive datasets. In addition, changes in the strength and spatial extent of AI detections may provide indirect evidence of changing defect characteristics. For example, while sub-pixel changes in crack width cannot be measured directly, an increase in the consistency or extent of crack detections between surveys may indicate widening or progression of an existing feature. Such observations would not constitute a measurement in themselves, but could be used to highlight areas requiring further engineering review.

Photographic Data



AssetScan Outputs



Figure 5. Example of orthomosaic image and identified defects at Block 41.

Automated Change Detection

The principal value of the baseline survey lies in establishing a repeatable photographic record against which future inspections can be compared.

Historically, comparison of gallery condition relied upon engineering observations and historic records, making comprehensive assessment of large structures both time consuming and subjective.

By combining repeatable image capture, photogrammetric processing and AI-assisted defect identification, future surveys can be compared directly against the baseline dataset. Existing defects can be assessed for changes in extent, continuity and associated staining or calcite deposition, whilst newly identified features can be highlighted automatically for engineering review. This provides a structured and auditable means of assessing change across the entire gallery rather than limiting assessment to previously recorded areas of interest.

The workflow adopted for Mullardoch is intended to treat change detection as an engineering screening exercise rather than an automated decision-making process. Potential changes identified by the software would be reviewed against the orthomosaics from both survey epochs, allowing engineers to rapidly determine whether a reported change is genuine or the result of variations in image quality, lighting conditions or other environmental factors. This approach retains engineering judgement at the centre of the assessment process whilst significantly reducing the volume of data requiring detailed review.

CONCLUSION

Defect Mapping for Mullardoch Dam

Overall, the survey met the requirements of the direction issued under Section 51(3) of the Act and provided a comprehensive set of baseline survey drawings that can be used as a reference for future inspections.

The deliverables provide a practical means of monitoring the condition of the galleries and shafts between formal survey intervals. Areas of concern can be reviewed and compared against the baseline record should any changes be observed during routine inspections. Furthermore, the outputs can be readily used in both office and field environments without the need for specialist software, allowing defects to be reviewed, annotated and communicated efficiently.

In addition to recording the known areas of interest, the survey identified numerous smaller defects and areas of calcite deposition throughout the galleries and shafts. Whilst these features are not currently considered significant, their inclusion within the baseline record provides a means of identifying future changes that may otherwise be difficult to detect.

Capturing and assessing the conditions in concrete dam drainage galleries using novel camera technology and AI image processing techniques

Underlying Technologies

The pilot study demonstrated that modern camera systems, photogrammetric processing and AI-assisted review can be successfully combined to create a comprehensive and repeatable record of the condition of concrete dam galleries and shafts. Whilst bespoke AI training was undertaken for this application, the results suggest that similar approaches could be applied to other reservoir assets where large photographic datasets are generated.

The methodology was developed with repeatability as a primary objective, providing a pathway towards future AI-assisted change detection and long-term condition monitoring. However, the greatest value of the approach lies in the creation of a complete and auditable photographic baseline against which future engineering assessments can be undertaken.

REFERENCES

- Gosschalk, E. M., Hinks, J. L., Johnson, F. G., & Jarvis, R. M. (1991). *ICOLD Question 65, R.26 - Overcoming the Build-Up of Stresses, Cracking and Leakage in Mullardoch Dam*. Paris, France: ICOLD.
- Hewitt, M. (2023). *Mullardoch Reservoir - Report on an Inspection under the Reservoirs (Scotland) Act 2011*. Cambridge, UK: Mott Macdonald.



The British Dam Society

Permanent seepage monitoring using thermal measurements at Toddbrook Reservoir following extensive improvement works

JULIUS HENSCHER, Solexperts GmbH
AXEL FABRITIUS, Solexperts GmbH
BARBARA HEINEMANN, Solexperts GmbH
SIOBHÁN BUTLER, Canal & River Trust
ALISTAIR LOBBAN, Canal & River Trust
DAVID PRISK, Canal & River Trust

SYNOPSIS

This paper presents the installation and initial operation of a ground temperature monitoring system at Toddbrook Reservoir for the assessment of seepage conditions within the dam embankment. The system is based on the GTC Kappelmeyer® method and consists of permanent in situ temperature measurements from an array of vertical soundings installed along the downstream side of the dam core. Automated data transmission enables continuous processing, real-time monitoring, alarming, and estimation of potential seepage flow velocities. Reservoir water temperature and water level are additionally measured to support interpretation of the temperature data.

Monitoring depths were defined according to the dam geometry and generally extend to approximately 2m below the dam–foundation interface, allowing the detection of potential underflow and scouring processes. Initial measurements have established a reference thermal state for future seepage assessment following reservoir refilling. The paper presents the monitoring concept, system installation, and the most recent monitoring results.

TODDBROOK RESERVOIR REQUIREMENT FOR A SEEPAGE MONITORING SYSTEM

As an additional reservoir safety improvement, during the extensive works to replace the spillway at Toddbrook Reservoir, the Canal & River Trust installed a temperature monitoring system within the embankment dam. This system is able to detect early signs of internal erosion, i.e. signs of increased seepage flow, to enable precautionary works to be commenced if required.

Permanent seepage monitoring using thermal measurements at Toddbrook Reservoir following extensive improvement works

Toddbrook Reservoir stores 1.2 million m³ of water to supply the canal system, feeding the Peak Forest Canal, the Macclesfield Canal and via these the Trent and Mersey Canal. This water enables boats to pass up and down the lock flights across the north of England and keeps nature thriving along the canal network. The reservoir was initially constructed in 1840, has been improved and modernised in many stages, and remains critical to keeping the canal system operating.

The reservoir suffered damage in 2019 following a significant rainfall event which caused damage to the auxiliary spillway. This spillway has now been replaced. The reservoir was held at 12m below the top water level as a precaution during the six years required to investigate, analyse, outline design, planning permission, detailed design and construct the improved spillway system.

While there is no evidence of ongoing internal erosion to the embankment, as a precautionary measure, the reservoir Inspecting Engineer supervising the statutory reservoir safety improvement works required seepage monitoring to be installed. The reservoir has been drawn down for a significant period of time and the spillway damage did expose the core of the dam. While neither are expected to cause internal erosion, it is prudent to install a monitoring system.

The Canal & River Trust compared a number of alternative geophysical systems and found the temperature sensing system provided by Solexperts (GTC Kappelmeyer®), and suggested by the Inspecting Engineer, to both be more cost effective and to have a proven success rate at identifying anomalies. The Trust has used this temperature system many times across the canal network to good effect; although the system at Toddbrook is the first to be permanent and to provide continuous monitoring.

The temperature monitoring system is now installed and being used to monitor the embankment as the reservoir is re-filled.

THERMAL LEAKAGE DETECTION METHOD

Surface water and near-surface ground temperatures exhibit seasonal variations, with temperature maxima and minima occurring at similar times. However, due to the low thermal conductivity and heat capacity of the ground, temperature fluctuations decrease with depth and show a phase shift relative to reservoir water temperatures. These thermal characteristics enable the use of temperature measurements for detecting seepage and subsurface fluid flow in dams and embankments (Kappelmeyer, 1997, Dornstädter and Heinemann, 2012).

Fluid flow through the ground causes temperature anomalies when infiltrating water differs in temperature from the surrounding ground. Advective heat transport associated with seepage modifies the dam's internal temperature field, making water temperature an effective tracer for locating leaks and zones of increased permeability. In summer, seepage zones are characterized by positive temperature anomalies, whereas in winter they show negative anomalies. During spring and autumn, when temperature contrasts are smaller, local deviations from the surrounding ground temperature can still indicate leakage (Armbruster et al, 1992).

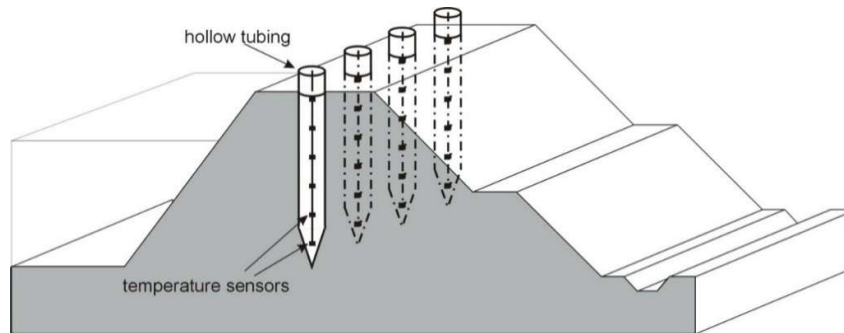


Figure 1 Method for ground temperature measurements – schematic sketch

To investigate these anomalies, GTC developed a thermometric measurement technique for loose sediments and embankments. Hollow pipes of small diameters are driven into the ground to depths exceeding 30m, and chains of temperature sensors at regular intervals are inserted as shown in Figure 1 (Armbruster et al., 1993). After thermal equilibration, temperatures are recorded at different depths, allowing seepage zones and their vertical and horizontal extent to be mapped. An idealised example illustrating the vertical extent of the seepage zone is shown in Figure 2a and Figure 2b. This measurement technique can be conducted as a one-time survey, periodically repeated campaign, or permanently installed monitoring system. Continuous temperature data acquisition, remote monitoring, and automatic interpretation of the acquired data are also possible (Dornstädter et al., 2017).

Permanent seepage monitoring using thermal measurements at Toddbrook Reservoir following extensive improvement works

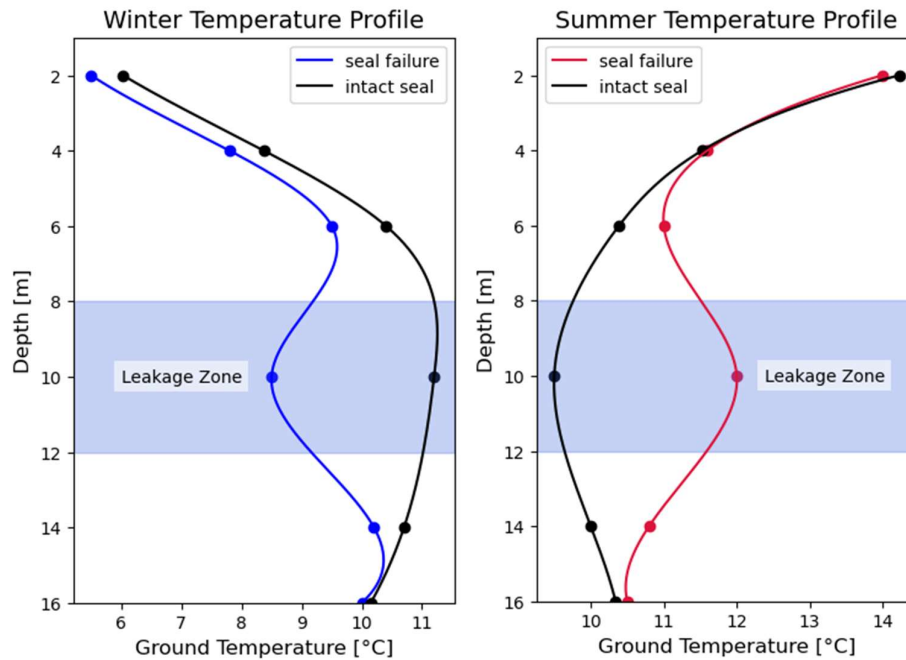


Figure 2a Ideal ground temperature distribution in winter

Figure 2b Ideal ground temperature distribution in summer

Over the last 25 years, this technique has been successfully applied to more than 500 km of dam sections, numerous lock chambers, and weirs. It has enabled the localisation of leaks, zones of increased permeability, and preferential flow paths within dam foundations, as well as the assessment of newly constructed or rehabilitated structures. Early identification of seepage zones is essential because prolonged seepage and internal erosion can significantly compromise dam stability.

Temperature monitoring can also be used to estimate seepage velocities. Variations in reservoir water temperature propagate into the dam with a measurable time delay that depends on flow velocity. By comparing temporal temperature changes in water and ground measurements, seepage velocities can be estimated using the phase shift between signals. Faster seepage transfers daily temperature fluctuations, whereas slower seepage is reflected mainly in seasonal variations, shown in Figure 3. By approximating the distance between the reservoir's dam interface and the point of measurements, pore velocities ranging from approximately 10^{-2} m/s to 10^{-6} m/s have been determined with satisfactory accuracy (Dornstädter and Heinemann, 2010).

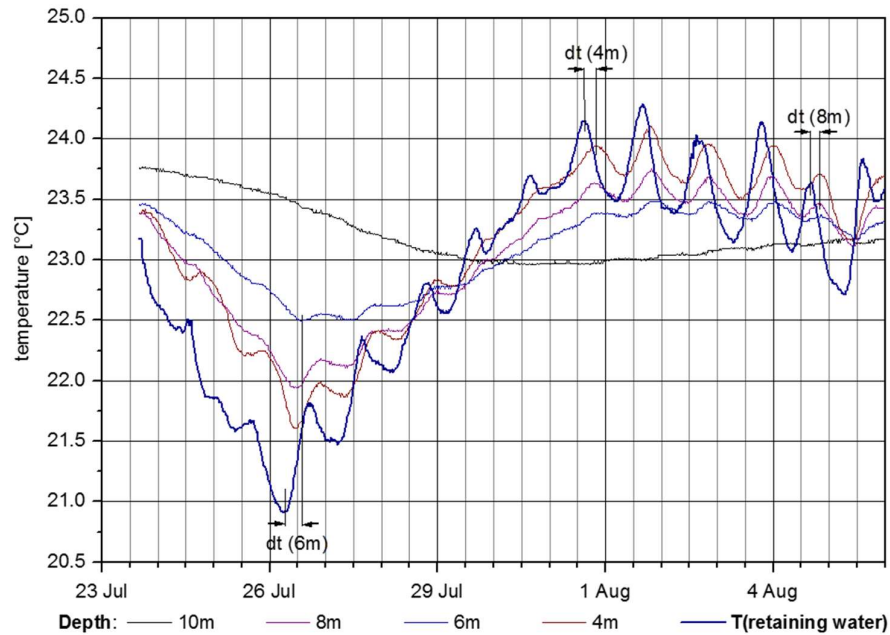


Figure 3 Temperatures over time show seasonal and daily fluctuations depending on flow velocities

MONITORING SYSTEM SPECIFICATIONS

Installation

Based on the theoretical framework outlined previously, a monitoring system was established at Toddbrook Reservoir.

The monitoring system at Toddbrook Reservoir consists of 27 hollow rods installed in the embankment downstream of the core. The locations of the temperature probes are shown in Figure 4, with a spacing of 10 m between probes. Probe depths were determined from the longitudinal section of the dam and range from 3m to 25m. Each probe is equipped with a temperature monitoring unit comprising a sensor chain with 1m sensor spacing and a connected battery-powered data logger. All units are housed in telemetry pillars (Fig. 5b and Fig 5c) for secure installation and operation on the dam. In addition, a temperature sensor chain and a water level sensor were installed to continuously monitor reservoir water temperature and water level.

Permanent seepage monitoring using thermal measurements at Toddbrook Reservoir following extensive improvement works

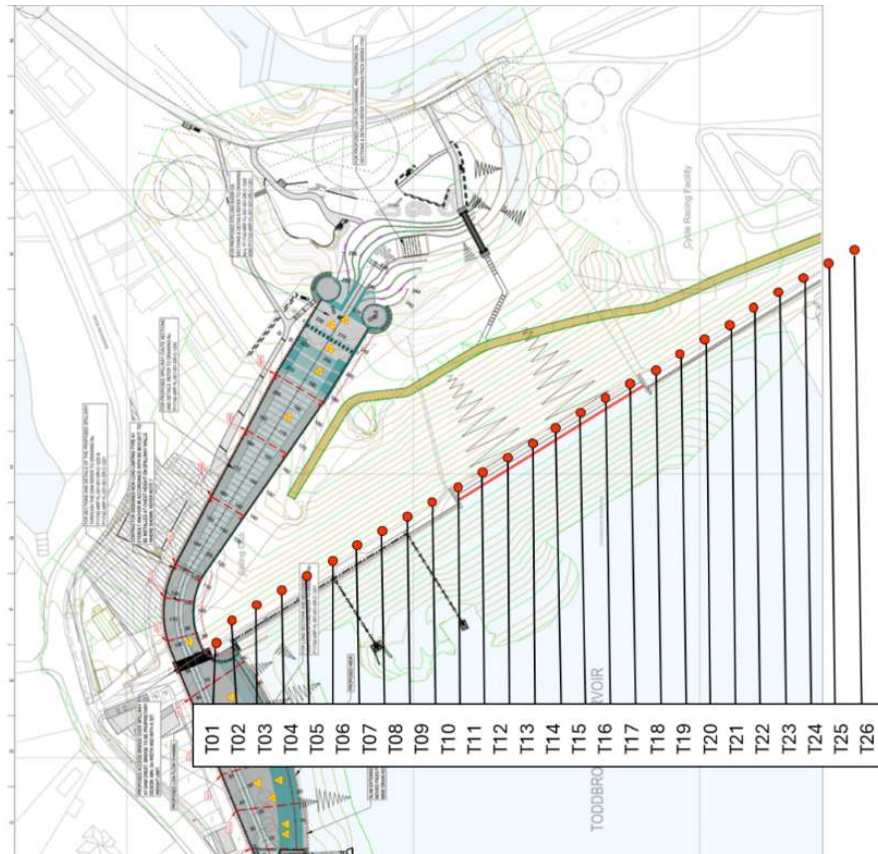


Figure 4 Location of temperature soundings

Data Acquisition

At the time of writing this paper, temperature measurements from all soundings are recorded at 10-minute intervals. The data are currently uploaded every 12 hours, although the upload frequency can easily be increased if required. Each logger transmits its data independently via a mobile data connection to a central server, where the data are processed and visualised automatically.

As each data logger operates independently the system includes a built-in redundancy, improving the overall reliability of the monitoring network.



Figure 5a Ramming of a temperature sounding steel tube at Toddbrook Reservoir



Figure 5b Data Logger stored in a telemetry pillar



Figure 5c Several temperature soundings, covered by telemetry pillars, along the downstream side of the dam crest

Alarm System

An automated alarm evaluation is implemented in the system by monitoring temperature changes. Different alarm types can be triggered by analysing temperature variations over different time intervals, e.g. over 1 day, 1 week, or 1 month. Evaluating temperature changes in different time intervals help to distinguish between different strength of seepage.

The initial alarm thresholds are based on temperature differences and are defined using preliminary assumptions and expected site conditions. After approximately one year of monitoring, these thresholds can be adjusted and refined according to the observed conditions and measured system behaviour. At this stage, additional alarm criteria based on automatic phase-

Permanent seepage monitoring using thermal measurements at Toddbrook Reservoir following extensive improvement works

shift calculations and seepage velocity estimations may also be implemented.

Seepage velocity estimations might also serve as alarm indicators. For example, seepage velocities exceeding 10^{-3} m/s may indicate the onset of potential internal erosion and trigger a red alarm. Velocities above 10^{-4} m/s may trigger an orange alarm, indicating that approximately one order of magnitude more response time remains. Additional thresholds may be defined at 10^{-5} m/s (yellow alarm) and 10^{-6} m/s (green alarm).

Data Visualisation

All current and historical measurement data, as well as alarm information, will be displayed on a dedicated website. The interactive platform provides a quick overview of the monitoring system status and enables users to access different types of visualisations.

These visualisations include alarm analysis, data display for different time intervals, 2D data visualisation, temperature–depth profiles, and the temporal development of temperatures within the dam and the reservoir, allowing instant identification and assessment of changes. Figure 6 illustrates the alarm status of the Toddbrook Reservoir on a specific day, demonstrating how operators can quickly obtain an overview of the current system status. A green status indicates that no alarm has been triggered, yellow indicates a warning condition requiring attention, and red indicates that further investigation is required.

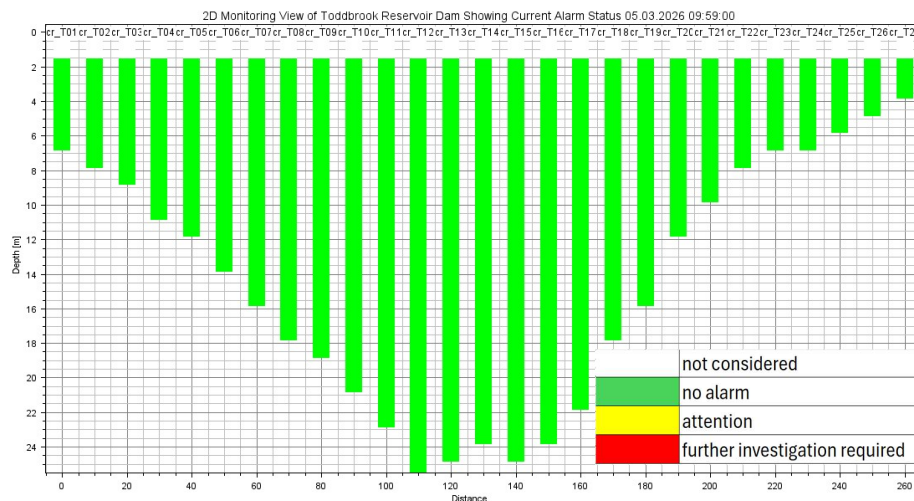


Figure 6 2D Longitudinal Alarming Profile, classification is based on temperature difference: green: < 1 K; yellow: ≥ 1.5 K; red: > 2 K (can change in future)

INITIAL RESULTS AND INTERPRETATION

The monitoring system is now functional and transmits data of ground and water temperatures to be evaluated. The reservoir was held close to empty during the restoration works, at a depth of 12m below top water level. As the water level is still low, the initial measurements do not show significant anomalies but provide a baseline prior to the refill operation. The reservoir is currently under a gradual refill, held at 10m below top water level at the time of writing.

The sensitivity of the monitoring system can be demonstrated at this pre-refill early stage by comparing sounding T09 to the other probes, as shown in the Figure 7a and 7b. While the remaining soundings exhibit nearly constant temperatures below approximately 13 m depth from the dam crest, T09 shows a gradual temperature increase at its three deepest sensors, located at depths between 16 m and 18 m, since the start of the measurements, as illustrated in Figure 7b. This slight deviation is most likely due to thermal conduction effects associated with reservoir flow through the nearby underground draw off pipe.

Permanent seepage monitoring using thermal measurements at Toddbrook Reservoir following extensive improvement works

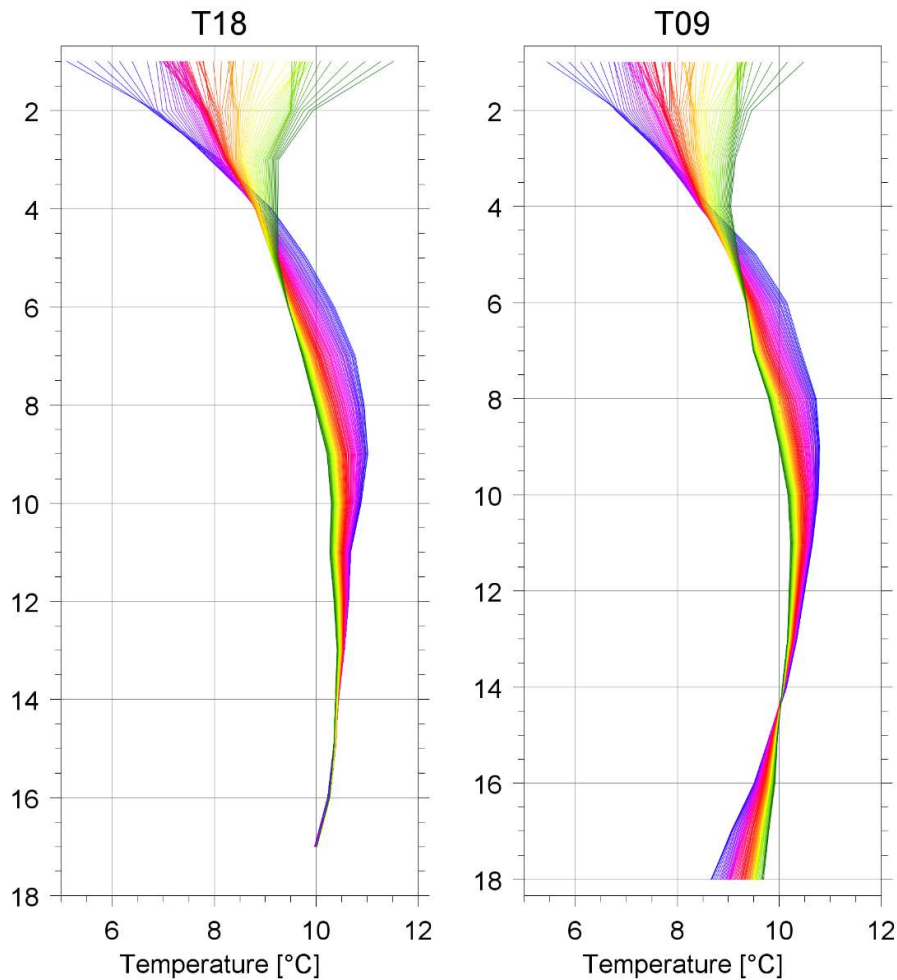


Figure 7a. Temperature–depth profiles from February 2026 (green) to May 2026 (blue) at sounding T18, showing an undisturbed temperature profile with visible seasonal temperature variations down to depths > 10 m.

Figure 7b. Temperature–depth profiles from February 2026 (green) to May 2026 (blue) at sounding T09, showing a minor anomaly related to the thermal influence of the draw-off pipe at depths from 16 to 18 m.

Out of all the soundings, the temperature evolution since the start of monitoring is illustrated in Figure 8 for probe T15, showing the ground temperature at depths between 3m and 24m below the surface, as well as the reservoir water temperature. In the case of a leakage, these temperature profiles would obviously converge towards the water temperature,

exhibiting a phase shift relative to it depending on the flow velocity (small phase shift – high flow velocity; large phase shift – low flow velocity).

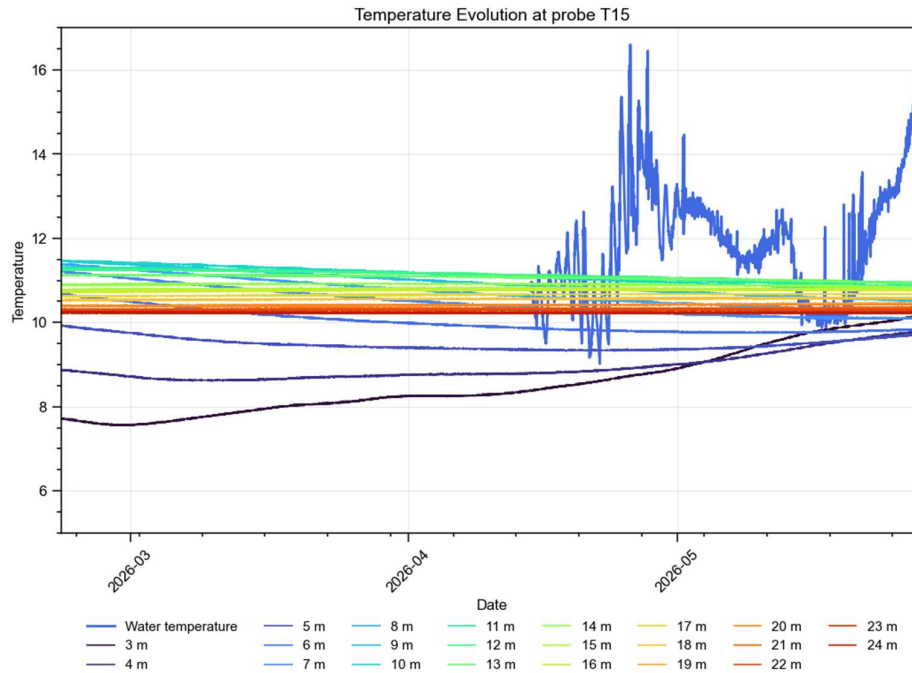


Figure 8. Temperature evolution at T15 over time measured by all sensors, covering depths from 1 m to 24 m.

Refilling of the reservoir began in April 2026 in a staged process, expected to be complete by late 2026. Future monitoring will show whether this change in boundary conditions leads to detectable variations in the ground temperature profiles.

OUTLOOK

By applying this retrofit thermal leakage detection method to the Toddbrook Reservoir, the dam is now equipped with a modern and sensitive monitoring system that enables detection of small changes in temperature within the dam and allowing an early detection of possible seepage water flow that might lead to internal erosion. This contributes to improving reservoir safety, ensuring the need for intervention can be recognised and communicated at an early stage.

**Permanent seepage monitoring using thermal measurements at
Toddbrook Reservoir following extensive improvement works**

REFERENCES

- Armbruster H, Dornstädter J, Kappelmeyer O and Tröger I (1993) Thermometrie zur Erfassung von Schwachstellen an Dämmen. *Wasserwirtschaft* **83(4)**.
- Dornstädter J and Heinemann B (2010) In situ detection of internal erosion, In Dam Safety. In *Sustainability in a Changing Environment*, Verlag der Technischen Universität Graz, Innsbruck (A). Presented at the 8th ICOLD European Club Symposium, pp. 463–467
- Dornstädter J and Heinemann B (2012) Temperature as tracer for in-situ detection of internal erosion. In *Proceedings of the 6th Int. Conference on Scour and Erosion*. Société Hydrotechnique de France, Paris (F), pp. 1369–1375.
- Dornstaedter J, Fabritius A and Heinemann B (2017) Online alarming for internal erosion: (paper and abstract). In *Proceedings of the 25th Meeting of the European Working Group on Internal Erosion* (EWG-IE 2017), Delft (NL), pp. 160–167.
- Kappelmeyer O (1957) The use of near-surface temperature measurements for discovering anomalies due to causes at depth. *Geophysical Prospecting* **5(3)**: 239–258, <https://doi.org/10.1111/j.1365-2478.1957.tb01431.x>.



The British Dam Society

Revitalising Resilience: Modernising 17 Service Reservoirs for a Sustainable Future

DAVID BELL, AECOM
SEÁNA KNOX, AECOM

SYNOPSIS

The refurbishment of seventeen service reservoirs, each comprising multiple reservoir cells, was a strategically planned project aimed at modernising critical water infrastructure while ensuring minimal disruption to water supply. The works addressed ageing components and incorporated advanced engineering solutions to enhance operational safety, efficiency, and reliability across all sites. Key elements of the refurbishment included the provision of new overflow arrangements to safely discharge excess water during high-flow conditions. Computational hydraulic modelling refined through-flow patterns in each cell, curbing short-circuiting and securing compliance with water quality regulations and enhanced the reservoirs' operational performance. Concrete repair strategies were implemented to address structural degradation, extending the service life of the reservoirs while maintaining their integrity. Valve refurbishment restored functionality and improved flow control, while access improvements enhanced safety and facilitated future maintenance activities. Operational risk assessments and safety planning ensured safe access, including controlled isolation and alarm management.

The works were undertaken in a programmed manner, with phased construction schedules designed to ensure uninterrupted water supply throughout the project. This approach minimised disruption to customers and maintained the reservoirs' role in meeting water demand. The complexity of working across multiple reservoir cells at each site required careful co-ordination to balance refurbishment activities with ongoing operational needs. This refurbishment project exemplifies a comprehensive approach to upgrading service reservoirs, balancing structural repairs, hydraulic optimisation, technological advancements, and operational continuity. By addressing both immediate and long-term needs, the project significantly improved the resilience, safety, and efficiency of the reservoirs.

INTRODUCTION

Service reservoirs are often unnoticed within the landscape, yet they form a critical component of Northern Ireland's water supply system, and their

Revitalising Resilience: Modernising 17 Service Reservoirs for a Sustainable Future

importance to public health and water security cannot be overstated. As these assets age, the need for timely refurbishment becomes essential to preserve their integrity and ensure continued resilience and regulatory compliance.

Delivering remedial works on live service reservoirs is inherently challenging; limited outage windows, unconfirmed internal conditions and confined space constraints make delivery complex. Coordinating alarms, isolations, materials, and specialist equipment must be achieved within narrow operational windows, all while responding to weather constraints and maintaining supply to customers. This paper describes the ongoing approach taken to deliver a multi-site refurbishment programme across seventeen service reservoirs in Northern Ireland, as indicated in Figure 1.

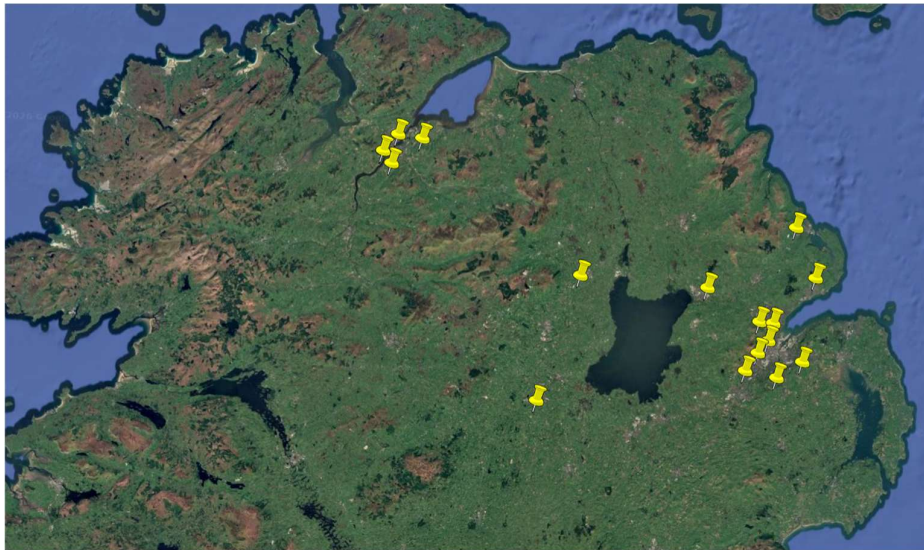


Figure 1. The 17 Service Reservoir Locations across Northern Ireland (Google Earth, 2026)

It highlights the technical, logistical, and operational challenges encountered, and the solutions developed to upgrade these critical assets while keeping them in service.

WHO, WHAT, WHERE, WHEN, WHY AND HOW?

Routine Section 10-type inspections undertaken since 2021 identified structural, hydraulic, and operational deficiencies across several service reservoirs in Northern Ireland (NI) Water's portfolio. As continuously operated assets, deterioration is expected; timely intervention is therefore essential to maintain integrity, resilience, and compliance ahead of forthcoming reservoir safety legislation. In response, NI Water launched a

remedial works programme covering seventeen service reservoirs across Northern Ireland. Each site comprises multiple cells and serves critical supply zones, making the programme both operationally sensitive and strategically important. Works remain ongoing, with five sites completed or nearing completion. AECOM is delivering design, contract administration, and site supervision; BSG is the contractor and NI Water is the client.

The scope varies by site but typically includes:

- Concrete repairs, external waterproofing, and drainage improvements.
- Valve refurbishment, pipework preparation and painting, and installation of column markers.
- Updates to on-site plans and Prescribed Forms of Record (PFORs).
- New overflow arrangements where required.

A whole-system refurbishment strategy was adopted to treat each reservoir as an integrated asset rather than pursuing isolated repairs.

Programme delivery is structured to safeguard supply. No two reservoirs within the same supply area are taken out of service simultaneously. While external works proceed concurrently, internal works requiring cell drain-down are carefully sequenced. Sites were programmed geographically to ensure only one cell within a supply zone is offline at any time, maintaining uninterrupted service. This approach allows several sites to run in parallel and better manages weather-dependent, time-consuming activities such as roof membrane installation.

Active communication has underpinned delivery. The project team coordinates regularly with NI Water's capital delivery, operational, reservoir safety, and asset management teams, as well as with the contractor. Early engagement on site readiness, operational constraints, and opportunities to combine works with other planned activities has driven efficiency and minimised disruption across the programme.

STANDARDISING NORTHERN IRELAND'S RESERVOIRS

Upcoming reservoir safety legislation in Northern Ireland, building on the principles of the Reservoirs Act 1975, has accelerated the programme delivery. While many of the interventions within this project address site-specific defects, an equally important objective has been to introduce a consistent, high-quality baseline across all seventeen reservoirs. The approach has been adopted to improve overall reservoir safety and ensure that operational staff encounter familiar arrangements regardless of location, reducing variability across the network and enabling more efficient maintenance, training and operational response.

Revitalising Resilience: Modernising 17 Service Reservoirs for a Sustainable Future

Several measures were introduced to achieve this; access ladders were upgraded to non-slip designs to improve health and safety during entry, while column markers were installed within each cell to support future inspections. These simple, low-cost additions significantly improve navigability within large, repetitive internal layouts where orientation can be challenging even with drawings. Pipework painting was also standardised to facilitate ease of asset identification; red for inlet arrangements, blue for outlet and green for scour and overflow.



Figure 2. Painted Valve work in Valve Chamber



Figure 3. Internal Pipework Painting and Signage including Column Markers

In addition to physical upgrades, the programme also sought to improve the consistency and quality of operational documentation. Updated PFORs and On-Site Plans were prepared for each reservoir to ensure that emergency procedures and instrumentation records were accurate and readily available.

AN ENGINEERING CHALLENGE: OVERFLOW DESIGN

Upgrading overflow arrangements formed one of the most technically challenging elements of the programme. The work considered two aspects:

1. assessing whether existing overflows could safely convey emergency flows; and
2. designing new enclosed overflow pipework at four sites where high-level slot overflows discharged directly into the valve chamber.

Existing Overflow and Scour Capacity Assessments

Routine S10-type inspections highlighted uncertainty around the capacity and condition of existing overflow and scour pipework. Several reservoirs had limited or incomplete record drawings, with limited documented information on the configuration of buried or internal pipework.

The review process commenced with the collection of all available information from the client's ArcGIS asset database to determine pipe sizes, materials, and fittings. This data was further enhanced by on-site surveys to confirm pipework routes and to review record drawings (when available).

Historic operational data including inflow, outflow, reservoir level and alarm status was obtained for each site as shown in Figure 4. This allowed the modelling to reflect real operating conditions rather than theoretical assumptions.

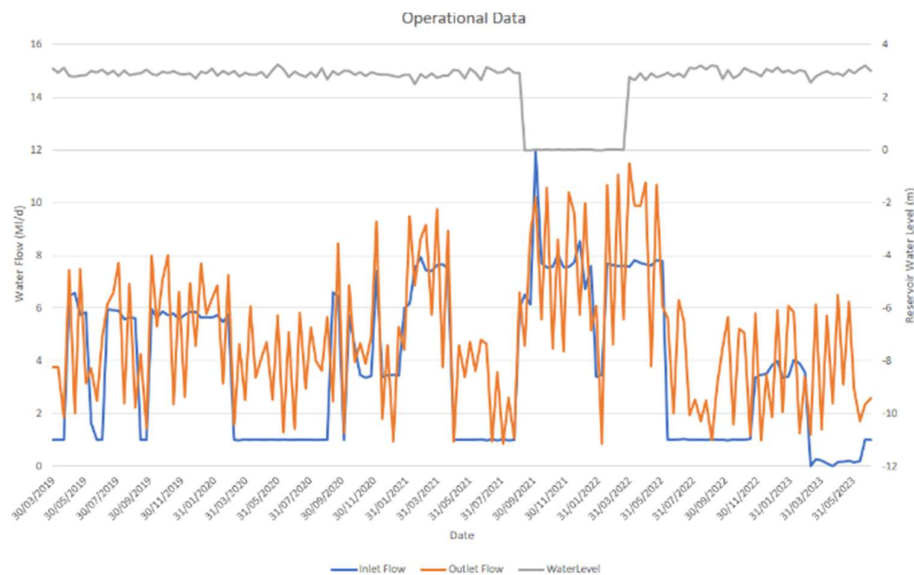


Figure 4. Operational Data Graph

A PipeFlow model, such as that shown in Figure 5, was developed to evaluate whether each reservoir could safely pass maximum inflow through the overflow system. A separate assessment was made for discharge capacity via the scour arrangements. The software facilitated the calculation of velocities, pressures, and potential restrictions. Conservative assumptions were employed to enhance robustness, including extreme inflow rates and elevated pipework roughness values to account for the condition of ageing assets.

Revitalising Resilience: Modernising 17 Service Reservoirs for a Sustainable Future

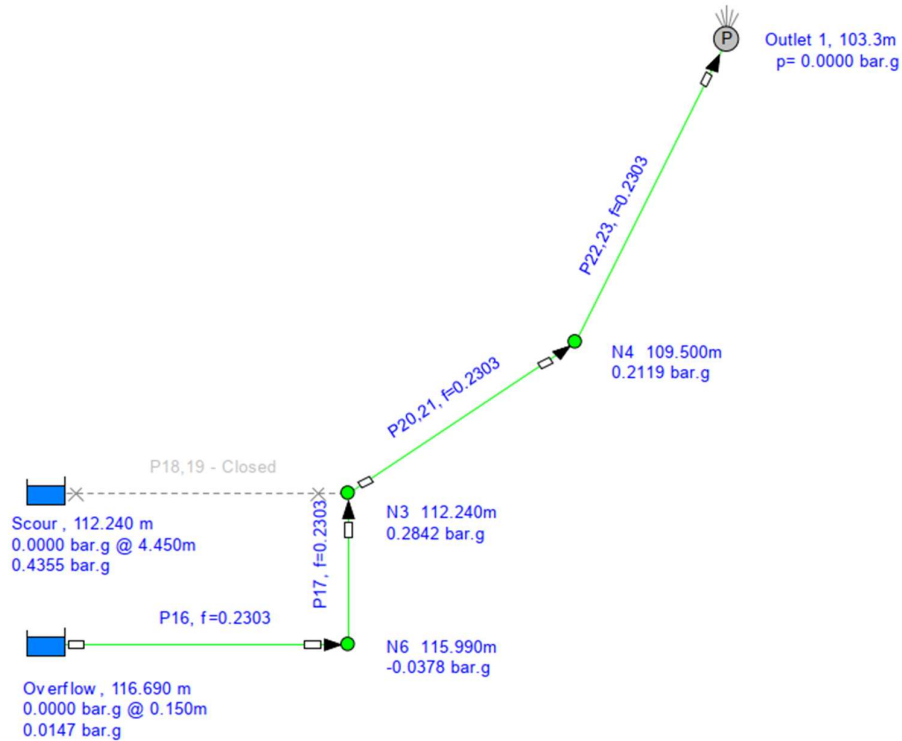


Figure 5. PipeFlow Results Image (Pipeflow Software, 2024)

Overflow Redesign

Four reservoirs required a redesign of their overflow arrangements, in response to the following recommendation under Measures to be taken in the interests of safety under section 10(3)(c) of the Act:

Following confirmation of the overflow arrangements, implement remedial works as necessary to ensure the safe and controlled discharge of overflows to enable full access to the valve chamber during all water levels.

These sites included high-level slot overflows that discharged directly into the valve chamber, as shown in Figure 6, before flowing into the downstream scour system through an opening in the invert of the valve chamber. The arrangement resulted in exposure to high chlorine levels within the valve chamber, risks of water ingress into electrical equipment, increased structural deterioration of valves, pipework, operating mechanisms and access arrangements caused by prolonged exposure to moisture.

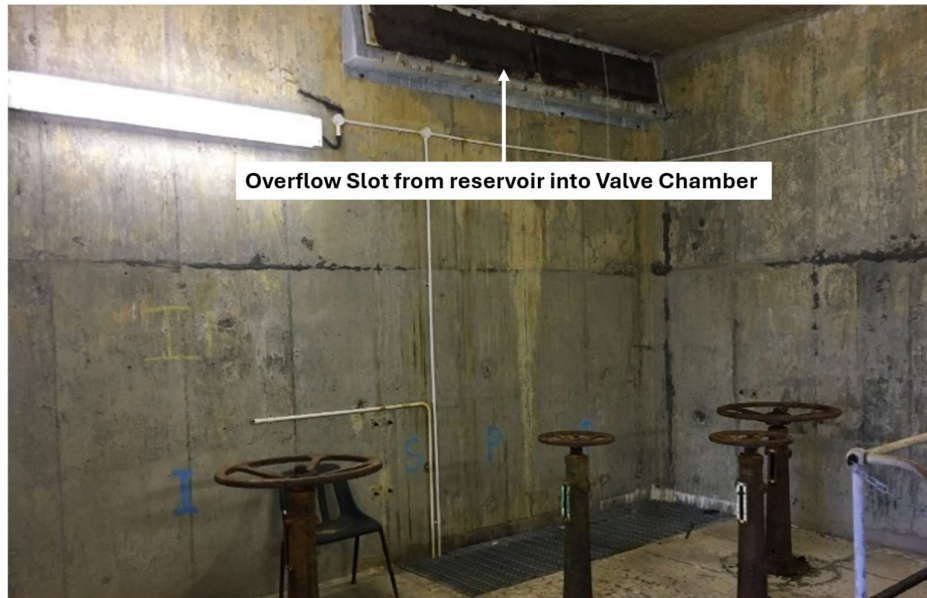


Figure 6. Existing High-Level Overflow Slots within Valve Chamber

3D laser scans of the valve chambers were conducted to capture precise geometry and identify spatial constraints, with the proposed pipework layouts modelled in Revit software to understand constructability issues and confirm compatibility with existing assets, supplemented by site visits and early engagement with the contractor. This collaborative approach was essential, given the confined nature of the chambers, where manoeuvring pipework and fittings is challenging and weight limits must be carefully managed.



Figure 7. 3D Laser Scan

Revitalising Resilience: Modernising 17 Service Reservoirs for a Sustainable Future

Two of the four sites were designed with all additional overflow pipework located within the valve chamber, while the remaining two required external routing to connect to the existing scour pipework. The internal solutions eliminated the need for excavation but demanded meticulous planning for installation in confined spaces. In contrast, the external solutions provided more physical space but introduced additional challenges, such as excavation near the reservoir structure, managing unrecorded buried utilities, and ensuring embankment stability. During the installation of the first external overflow, excavation uncovered an unrecorded pipe at the proposed connection point to the downstream scour system, requiring swift redesign of the downstream alignment to prevent delays and maintain operational supply. The importance of effective and timely communication among the contractor, designer, and site operatives was highlighted to facilitate resolution, enabling timely procurement of materials and allowing work to proceed without significant disruption.



Figure 8. External Overflow Installation

At the time of writing, one external overflow has been successfully completed, and installation of the first internal overflow is underway.

AN ENGINEERING CHALLENGE: ROOF DRAINAGE

Several of the reservoirs required substantial upgrades to their roof waterproofing and drainage systems, to ensure the watertightness of the roof slab. While the scope was less technically challenging, these works proved among the most logistically complex due to the limited structural capacity of the roof slabs and weather-dependent working windows. The refurbishment was sequenced to include controlled removal of overburden/topsoil, installation of a multi-layer waterproofing system, reinstatement of drainage pathways, and replacement of filter media and topsoil. Throughout the works, construction loading was managed rigorously to avoid overstressing the roof slab, which was subject to strict imposed load limits.

The new waterproofing system comprised an impermeable membrane, a protective layer, and a drainage membrane, followed by the reinstatement of stone and topsoil as shown in Figure 9. New drainage was also installed around the perimeter of each roof to minimise the risk of infiltration into the reservoir structure.



Figure 9. Membrane Installation

The installation of the membrane required dry and calm weather conditions to ensure the system's integrity while safeguarding site personnel during the process. This necessitated close coordination between the contractor and

Revitalising Resilience: Modernising 17 Service Reservoirs for a Sustainable Future

site operatives to schedule work around favourable weather forecasts and maintain progress across multiple sites.

At several sites, stripping the roof overburden exposed degraded membranes, failed bandages, and previously unrecorded elements such as the central roof trough. These findings required close collaboration among the project team to efficiently implement suitable solutions.

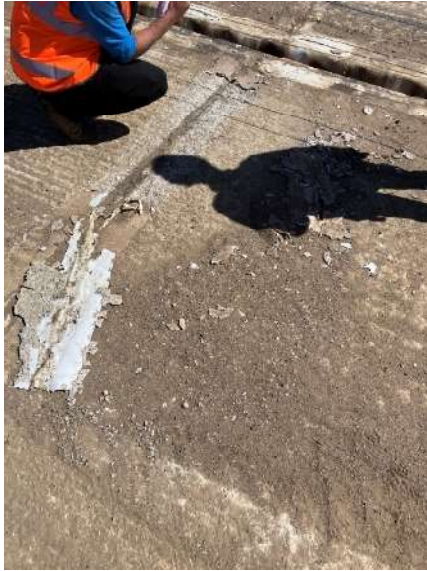


Figure 10. Existing Failed Bandages



Figure 11. Exposed Central Roof Trough

The works also required careful coordination with other site users. Existing paths had to be reinstated and electrical or telemetry cables rerouted where they conflicted with excavation or reinstatement activities. These challenges highlighted the importance of flexible design, proactive communication and a strong working relationship between the contractor, designer and site operatives.

EXPECTING THE UNEXPECTED

Working within live service reservoirs requires a constant readiness to adapt. While "expect the unexpected" may be a cliché, it became a defining theme of this programme. The combination of ageing assets, incomplete records, weather-dependency, and operational constraints meant that even well-planned activities often necessitated reassessment and redesign.

One of the most significant external challenges was the weather. Northern Ireland experienced its wettest January in 150 years (BBC News, 2024), which severely impacted the timelines for external works, including excavation and membrane installation.

Deteriorating conditions were also noted due to the time elapsed between the initial inspection and construction. At most reservoirs, nearly five years had passed since the S10-type inspection. Once drained, internal cells revealed increased deterioration in key elements such as structural concrete, columns, baffles, and jointing. These findings highlighted the critical need for early access to the cells to facilitate a joint survey by the design team and contractor, allowing for collaborative refinement of the scope of works.

To mitigate the impact of deteriorating conditions on the project schedule, a scope confirmation survey was conducted immediately following the draining of the cell. This process enabled prompt decision-making, accurate material ordering, and clear agreement on the scope of repairs. Effective communication from the project team was essential throughout this process to minimise delays. A 360° camera survey was also conducted to document the condition of the cell both before and after the works.

CONCLUSION

The introduction of reservoir safety legislation in Northern Ireland will impose formal responsibilities on reservoir managers, including enhanced requirements for inspection, monitoring, and emergency preparedness. Therefore, establishing consistent standards across the service reservoir portfolio is crucial to support future compliance and alleviate the long-term burden of asset management. At the midpoint of this two-year programme, the advantages of a coordinated, standardised approach are already apparent. The refurbishment philosophy has prioritised extending asset life, enhancing operational safety, and addressing risks associated with ageing infrastructure.

While service reservoirs may seem straightforward in design, they are inherently high-risk environments due to confined space entry, chlorine hazards, and their critical role in maintaining water supply. The work completed to date has bolstered resilience across the network through structural repairs, improved overflow arrangements, enhanced external waterproofing, and standardised access solutions. The implementation of consistent design standards has also increased efficiency, reduced future maintenance demands, and fostered better operational consistency across multiple sites. Collectively, the enhancements made across these seventeen reservoirs mark a significant advancement toward a more resilient, compliant, and future-ready service reservoir portfolio for Northern Ireland.

Revitalising Resilience: Modernising 17 Service Reservoirs for a Sustainable Future

ACKNOWLEDGEMENTS

The authors wish to express their gratitude for the invaluable contributions of the entire project team, including the Client, NI Water (capital delivery, operational, reservoir safety, and asset management teams), and the Contractor, BSG Civil Engineering.

REFERENCES

- Google Earth (2026) *Satellite imagery of Northern Ireland*. Available at: <https://earth.google.com/web/> (Accessed: 19 May 2026).
- Pipeflow Software (2024) *Pipe Flow Expert – Pipe Flow Analysis & Hydraulic Modelling Software*. Available at: <https://www.pipeflow.com/pipe-flow-expert-software> (Accessed: 1 June 2026).
- BBC News (2024) *Northern Ireland records wettest January in 150 years*. Available at: <https://www.bbc.co.uk/news/articles/c4g42e5l6dro> (Accessed: 19 May 2026).



The British Dam Society

From Data Management to Integrated Dam Safety: DAMDATA, a Digital Platform Approach

S HOPPE, OFITECO, Madrid, Spain

J FLEITZ, OFITECO, Madrid, Spain

H C YARRITU-SANCHEZ, SICE, Reading, United Kingdom

SYNOPSIS

Efficient dam safety management involves the need for digital platforms capable of integrating technical, operational and regulatory information within a unified environment. DamData is a digital platform that has progressively evolved from a dam monitoring platform into a comprehensive dam safety and asset management system. It incorporates dedicated modules for manual and automatic dam monitoring, visual inspections, Emergency Action Plans (EAP), flood management, maintenance planning, centralised document control and digital twins. This integrated approach enhances traceability, consistency, and transparency across the full dam service lifecycle, from routine surveillance to emergency preparedness.

Operational experience demonstrates that consolidating safety-relevant information within a single digital platform improves decision-making, supports engineers and reservoir managers in maintaining regulatory compliance, and enhances operational efficiency. DamData illustrates how innovation and technology can move beyond data storage to become key enablers of a proactive, integrated dam safety management process.

INTRODUCTION

Efficient dam safety management involves multiple aspects, including evolving regulatory requirements, ageing infrastructure, knowledge transfer, and the increasing need for timely risk-informed decision-making. These challenges, combined with frequent limitations due to fragmented information and data series and discontinued safety inspections and operation and maintenance activities, have highlighted the need for digital platforms capable of integrating technical, operational and regulatory information within a unified environment.

DamData is a digital platform originally developed for the analysis of dam monitoring and instrumentation data. Over time it has progressively evolved into a comprehensive dam safety and asset management system. Today, DamData is deployed successfully at more than 400 dams worldwide and is

From Data Management to Integrated Dam Safety: DAMDATA, a Digital Platform Approach

currently being implemented for several portfolios of dams in Europe, America and Australia.

In support of a holistic dam safety approach, DamData incorporates dedicated modules for manual and automatic dam monitoring, visual inspections, Emergency Action Plans (EAP), flood management, maintenance planning, centralised document control and digital twins. This integrated approach enhances traceability, consistency, and transparency across the full dam service lifecycle, from routine surveillance to emergency preparedness.

Operational experience demonstrates that consolidating safety-relevant information within a single digital platform improves decision-making, supports engineers and reservoir managers in maintaining regulatory compliance, and enhances operational efficiency. DamData illustrates how innovation and technology can move beyond data storage to become key enablers of a proactive, integrated dam safety management process.

OBJECTIVE OF THE PLATFORM

The main objective of implementing the platform is to streamline all information related to dam safety management, taking advantage of the global digital transformation process and the latest developments in IT technology to optimise dam safety management processes. A key element of that process is the implementation of the Building Information Modelling (BIM) methodology.

The platform's objective is not limited to creating a mere data repository for dam safety, nor only to providing a common visualisation system. Both capabilities would already represent a significant improvement compared with the current situation, but the goal pursued is broader: to obtain, in real time, higher-level information that constitutes a useful tool for decision-making related to the management of dam operations under normal, extraordinary and emergency conditions.

That enables, in addition to unifying and standardising information and workflows of O&M works, agile surveillance and monitoring of dam behaviour to facilitate decision-making.

PLATFORM DESCRIPTION

DAMDATA was originally developed as a comprehensive platform for the effective management of monitoring data. During its implementation for different dam owners, it was enhanced with additional functionalities and integrated software solutions, evolving into a fully comprehensive

monitoring system. Its modular architecture allows for customised adaptation to the specific requirements of each dam owner.

In this chapter, the different modules are described.

Administration module

The platform provides a centralised access interface and user management system. User authentication is performed through a secure verification process tailored to the IT requirements of each dam owner.

It serves as the management core of the system, enabling the administration of the different modules and applications. The platform is fully customisable, allowing users with administrative privileges to configure the navigation structure, define displayed options, and manage user profiles and access permissions.

Permissions can be assigned individually to each user and infrastructure component, with clearly defined read and write access rights.

Technical data sheet

This module provides access to the general information of each dam, which can be viewed and edited by the user.

The data sheet comprises approximately 80 to 100 fields, depending on the specific characteristics of the dam. Each field is associated with a defined data type, enabling efficient data validation, preventing incorrect data entry, and facilitating faster searches.

Data can be added or updated either manually or through data import processes. All fields—including images—as well as the associated sections are fully configurable, allowing flexible structuring of the information.

Advanced search and filtering functions can be applied across all data fields. In addition, the data sheet can be exported for printing, and the integrated image gallery allows visualisation of photographs in enlarged format.

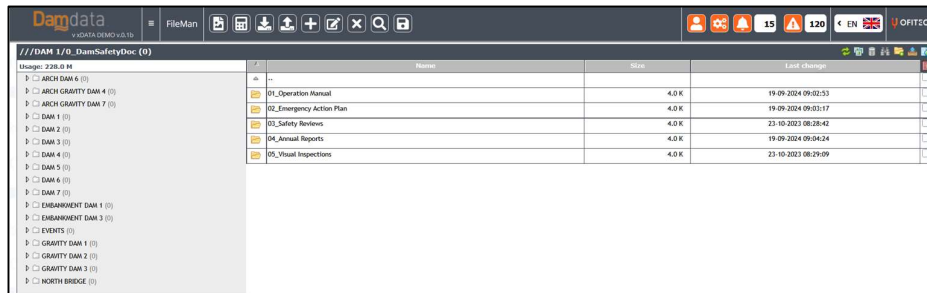
Technical archive

The Technical Archive module enables the storage and management of a wide range of documents and files. It incorporates a document management system based on a hierarchical folder structure, facilitating efficient classification, organisation, and protection of information (see figure 1).

The standard file operations include the creation, deletion, modification, sorting, and filtering of both folders and documents.

From Data Management to Integrated Dam Safety: DAMDATA, a Digital Platform Approach

The module also provides advanced search capabilities, allowing users to define specific keywords and select the fields in which the search is to be performed.



Document	Size	Last Change
01_Operation Manual	4.0 K	19-09-2024 09:02:53
02_Emergency Action Plan	4.0 K	19-09-2024 09:03:17
03_Safety Reviews	4.0 K	23-10-2023 08:28:42
04_Annual Reports	4.0 K	19-09-2024 09:04:24
05_Visual Inspections	4.0 K	23-10-2023 08:29:09

Figure 1. Document Manager for the Technical Archive

Emergency Action Plan

This module serves as a management tool for the Emergency Action Plan (EAP), supporting decision-making processes for the activation and management of emergency situations.

Integrated with the monitoring module, the system processes monitoring data and flood indicators to perform a threshold-based analysis, automatically suggesting the corresponding emergency scenario.

Once a specific scenario is triggered based on defined indicators, the system clearly and concisely presents the associated actions established in the Emergency Action Plan. These include communication and notification procedures, surveillance and control tasks, and the implementation of corrective measures.

For communication and notification actions (email, fax, and telephone), the system incorporates predefined contact lists and message templates. These can be executed automatically through structured forms, ensuring full traceability of all procedures in compliance with applicable regulations. In this way, the operator is supported throughout all phases of the emergency management process.

For surveillance, control, and corrective actions, the module also integrates the corresponding procedures, providing clear guidance for their implementation in accordance with the Emergency Action Plan.

Monitoring and surveillance

This module provides comprehensive management of monitoring data and visual inspections. It includes different tools for the proper implementation

and monitoring of any coordinated instrumentation plan and any periodic inspections that are required. It covers the processes of acquisition, processing, storage, management and evaluation of monitoring data to ensure continuous dam control, as well as detailed behaviour analysis.



Figure 2. Example of dashboard

The dashboard (see figure 2) summarises information in a visual way and displays sensor readings superimposed on drawings or images. Users can create or modify dashboards by combining external content with elements stored in the system database.

The dashboard is directly linked to other platform functionalities and also serves as an intuitive navigation interface, enabling quick access to detailed information within the monitoring module.

The included calculations tool allows the user to setup any formula to convert raw data to engineering values.

The system allows the setup and storage of individualised multivariable charts and report layouts (see figure 3) for helping the visualisation and interpretation of data.

From Data Management to Integrated Dam Safety: DAMDATA, a Digital Platform Approach



Figure 3: Example of the report layout

Mobile application for data reading and inspections

Manual data entry is carried out using the DAMDATA application for mobile devices. It is specifically developed to support operators in tasks such as entering readings, checking thresholds and verifying data quality (see figure 4). It synchronises with the platform to ensure proper control of new data entered into the system.

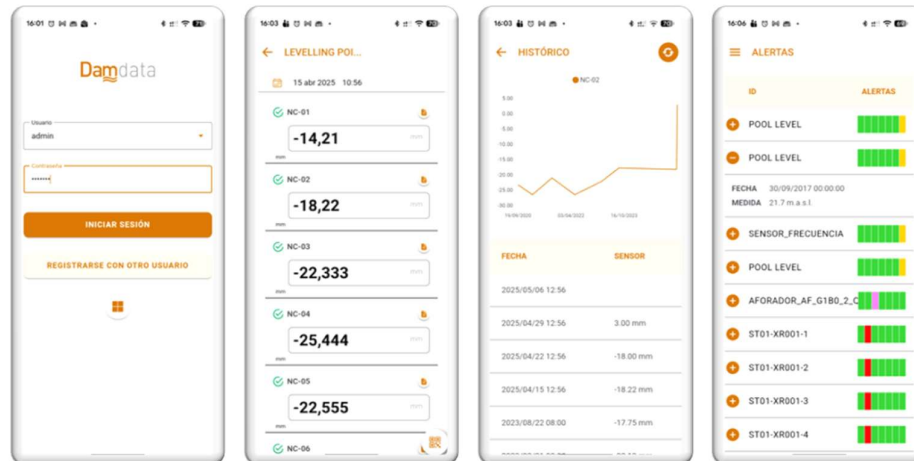


Figure 4: Example of the mobile application

As being fully integrated into the platform, the platform manager allows fieldwork planning, resource assignment and control of completed, planned and pending routes.

The Visual Inspections module enables the creation of customised inspection forms tailored to specific inspection activities. It also supports the

centralised management of these forms and their corresponding results within the platform.

Maintenance Plan

Through this module, maintenance tasks established in the O&M Plan are managed and monitored. It is organised through a tree structure that allows selecting infrastructures and/or elements to query and modify.

It includes an inventory of elements to be maintained with all information required for identification and carrying out O&M works.

It provides a planning tool showing upcoming and completed works as well as ongoing and closed incidents in tabular view. Charts and statistics support the management of the generated information.

The generation of reports facilitate the analysis of elements, work orders and incidents.

Reservoir and flood management

The reservoir and flood management application's main objective is to provide essential and updated information for dam operation during flood events. It summarises information and technical properties of outlet works as well as characteristic reservoir levels.

The incorporated discharge calculator provides the outflow as a function of the reservoir level or vice versa and calculates the required opening of each spillway gate and outlet valve.

Different inflow hydrographs can be defined, edited, imported, deleted and visualised. All simulation data can be saved and shown in the Simulation History and exported to pdf files.

BIM

The Building Information Modelling (BIM) module is conceived as a tool for infrastructure management based on BIM methodology, centralising all relevant information within a digital information model.

The BIM viewer is the main component of the module, allowing interactive visualisation of three-dimensional models in a virtual environment (Figure 5). It includes a wide range of integrated tools and features and is designed to be intuitive and user-friendly.

The navigation panel provides hierarchical tree structures of model elements grouped by modules, elements, and classes, enabling users to navigate and select specific elements for dedicated actions. Management of IFC models is also supported.

From Data Management to Integrated Dam Safety: DAMDATA, a Digital Platform Approach

The BIM module can interact with other platform modules—such as Instrumentation, Technical Archive, and Maintenance—allowing users to link model elements with additional information from these modules or from external sources, thereby facilitating efficient access to relevant infrastructure data.

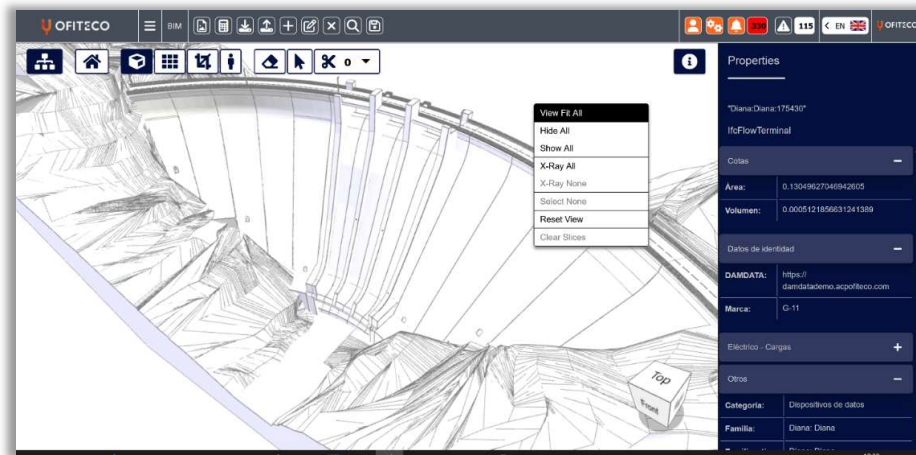


Figure 5: BIM Viewer

LESSONS LEARNT FROM RECENT IMPLEMENTATIONS

The key aspect of the implementation process is the acceptance by the different users. The process should be user-friendly and helpful for the different specialists involved in surveillance and O&M tasks. Therefore, a software solution that had already been approved by different public and private dam owners avoids a long testing process and reduces software developments to a minimum.

The second is the establishment of a detailed and flexible training plan that can cope with all users with different professional background and a high variety of user cases, such as surveillance (data reading, evaluation and behaviour analysis), maintenance (inspection, testing and repairing of civil, electrical and mechanical installations) and operation (flood and emergency management). A combination of online training with face to face training sessions was very helpful for the implementation process.

Owners usually operate dams with a broad variety in characteristics and age. This is also reflected in the diversity of the installed monitoring systems, outlet works and other equipment. Some installations are quite simple and aged, while others have been modernised and are completely automated. Different systems from local and international manufactures are usually

installed. This requires a flexible solution of the software to incorporate a complete inventory and all necessary working processes.

The organisation of technical documentation within the Technical Archive focuses on the homogenisation of information and naming of files. This work depends a lot on each local team due to the big differences in the quality and organisation of the available information. Documents are stored in a predefined structure of folders to facilitate easy access. Long or unclear filenames have to be modified and, in some cases, valuable information is only available in paper format and has to be digitised.

For the customisation and configuration of each dam, the involvement of the dam owner is crucial. He has to compile all relevant and updated information that can then be processed and adapted. During that process, obsolete and erroneous data can be detected in order to configure a representative database.

For the integration of BIM models, a close collaboration with the modeller is required to guarantee a correct integration of information. This is essential for establishing the same codification and incorporating the necessary information in the model.

CONCLUSIONS

The implementation of a dam safety platform enables the standardisation of information management and operational processes associated with dam safety and O&M activities. This contributes not only to enhanced safety levels, but also to the optimisation of available resources and the effective compliance with applicable Technical Standards for Dam Safety.

The use of an existing and validated software solution allows implementation timelines to be shortened, while minimising the need for additional software development. To ensure appropriate system configuration, the involvement of a multidisciplinary team with proven experience in dam safety and maintenance is essential. Such expertise is required to correctly interpret the available information and align the system with established operational procedures.



The British Dam Society

Implementing a National Risk-Informed Framework for Dam Safety: Lessons Learned from Spain

S HOPPE, OFITECO (Madrid, Spain)

H C YARRITU-SANCHEZ, SICE (United Kingdom, Spain)

M ESPLIEGO PASTOR, OFITECO (Madrid, Spain)

P PARRAS PAZ, OFITECO (Madrid, Spain)

I ESCUDER-BUENO, iPresas Risk Analysis (Valencia, Spain)

A MORALES-TORRES, iPresas Risk Analysis (Valencia, Spain)

JC DE CEA, MITECO (Madrid, Spain)

R CASTILLA, MITECO (Madrid, Spain)

F J ROMERO BERMÚDEZ, Guadalquivir River Basin Authority (Jaén, Spain)

SYNOPSIS

The General Water Directorate is currently implementing a nationwide risk-informed dam safety management programme in Spain, with the aim of strengthening governance, incorporating climate change considerations and prioritising investments. The methodology combines a semi-quantitative phase for failure mode identification with a quantitative phase for risk assessment.

This paper presents the results obtained in the Guadalquivir River Basin, where 55 dams and two irrigation ponds were analysed, yielding comparable risk profiles and prioritised risk reduction measures.

INTRODUCTION

Spain has a large and diverse dam portfolio, many of which were built during the twentieth century and are now affected by progressive ageing. This situation poses a major challenge for the coming years in terms of maintenance and safety, not only because of the age of the infrastructure, but also due to the evolution of regulations, the increase in safety standards, and the effects of climate change.

In this context, the General Water Directorate (DGA), the body responsible for the safety management of more than 300 state-owned dams in Spain, launched in 2021 a national initiative aimed at developing and implementing a risk-informed safety management system. This system considers both the current condition of dams and future scenarios influenced by climate change, with the objective of improving the objective and efficient prioritisation of dam safety investments.

Implementing a National Risk-Informed Framework for Dam Safety: Lessons Learned from Spain

This initiative was structured through two types of contracts: on the one hand, eight sub-contracts devoted to the execution of risk analyses in the different river basin districts, carried out by teams composed of qualified personnel and specialists from a wide range of technical fields related to dam safety; on the other hand, one contract specifically devoted to the overall coordination of the programme and the harmonisation of methodological criteria across the different sub-contracts.

This paper describes the structure, methodology and results of the risk analysis process applied to the 55 dams and two irrigation ponds in the Guadalquivir River Basin.

RISK ANALYSIS METHODOLOGY

The methodology applied for dam risk assessment follows the guidelines established in SPANCOLD Technical Guide P-8 (2012), which defines a structured process to determine the current and future condition of each dam, considering the impact of climate change and comparing the results with international risk tolerability criteria in order to contextualise the safety status of the infrastructure under analysis.

The process is mainly developed in two phases: a semi-quantitative phase and a quantitative phase. This is later followed by an evaluation of risk reduction measures aimed at lowering risk levels and optimising resource allocation.

Semi-quantitative phase

In this first phase, a multidisciplinary team of dam safety specialists carries out Potential Failure Mode Identification (PFMI) workshops for each dam. This requires a detailed understanding of the infrastructure, taking into account its construction characteristics, operating history and specific conditions. The procedure includes:

- **Review of information and site visit**, to gather information from the dam technical archive and assess its current condition. Historical documentation is reviewed and analysed together with more recent documents such as Operating Rules, General Safety Reviews, hydrological studies, Emergency Action Plans, and Annual Performance Reports.
- **Identification of potential failure modes** for each dam through participatory sessions based on the information gathered and the field inspection.

- **Semi-quantitative classification**, assigning a qualitative risk rating (probability and consequences) to each identified failure mode, in accordance with the ranges defined in Table 1.

Table 1. Ranges used for the semi-quantitative classification according to probability and consequences

Probability		Consequences	
Remote	$<10^{-6}$	C1	Minor economic damage
Low	$10^{-6} - 10^{-5}$	C2	Significant economic damage
Medium	$10^{-5} - 10^{-4}$	C3	<10 lives
High	$10^{-4} - 10^{-3}$	C4	10-100 lives
Very High	$>10^{-3}$	C5	100-1000 lives
		C6¹	>1000 lives

This phase provides an initial overview of the risk and safety condition of the dams analysed and forms the basis for the subsequent quantitative analysis.

Quantitative phase

In this second phase, the most relevant failure modes identified in the previous phase are analysed in greater detail. These failure modes are selected by a group of experts in order to ensure that the risk associated with each dam is adequately characterised. To this end, quantitative risk models are developed using event trees that allow the probability and consequences associated with each failure mode to be estimated. This provides a more precise and robust assessment than the semi-quantitative phase.

The risk models are mainly structured into three components:

- **Loads**, representing the external events that generate stresses on the dam and whose nature varies depending on the scenario analysed. These generally include inflow hydrographs, earthquakes, previous reservoir levels, outlet works reliability and routing studies. All loads are assigned a probability.
- **Response**, which estimates the probability of dam failure for the failure mode under analysis, using expert judgement or specific methodologies based on technical recommendations.

¹ The consequences C3, C4, C5 and C6 also entail economic damage, but loss of life prevails in their classification.

Implementing a National Risk-Informed Framework for Dam Safety: Lessons Learned from Spain

- **Consequences**, which estimate the economic and social damages resulting from dam failure. To this end, hydraulic simulations are carried out and tools such as LifeSim are used. In this way, hydraulic information is integrated with demographic data, infrastructure and land-use information, providing a detailed estimate of potential life loss and material damage.

Risk for each structure is obtained by combining probability and consequences. Two types of risk are distinguished:

- **Total risk**: the total downstream flood risk, including both cases where the dam fails and cases where it does not.
- **Incremental risk**: the portion of risk that is exclusively attributable to dam failure.

In addition, in order to incorporate the effect of climate change into the risk analysis, the models consider different timeframe scenarios that modify both the loads and the downstream consequence exposure. In general terms, climate change tends to increase risk due to the growing frequency of extreme events.

Overall, the quantitative phase adds rigour and reduces the subjectivity inherent in the semi-quantitative phase, thereby strengthening the reliability of the process and providing a solid basis for decision-making.

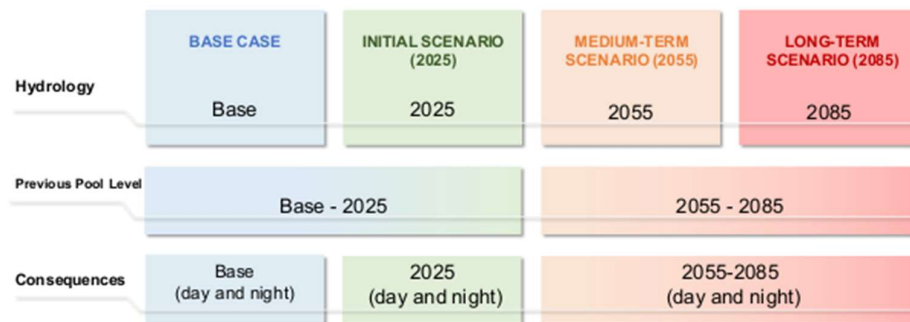


Figure 1. Scenarios considered in the risk model

Prioritisation of measures

Following the quantitative risk assessment, the results obtained are compared against international risk tolerability criteria recommended by organisations such as USBR, ANCOLD and USACE in order to assess the dams' safety status in risk terms.

- **Bureau of Reclamation (USBR)**: its recommendations are based on average risk values and establish a limit for annual probability of failure of 10^{-4} , together with a social risk limit below 10^3 lives per year.

- **Australian National Committee on Large Dams (ANCOLD):** its tolerability recommendations are based on the annual cumulative probability of exceeding a given level of consequences.
- **United States Army Corps of Engineers (USACE):** adopts ANCOLD limits, adding an additional restriction on the maximum tolerable consequences as well as an individual risk criterion, which must remain below 10^{-4} .

Based on this comparison, both, dams whose risk exceeds the recommended limits and those that comply with them, are identified. A set of measures is then defined to reduce risk in both the initial scenario (2025) and the medium-term scenario (2055), thus incorporating the effects of climate change into the decision-making process.

The proposed measures may include structural works, operational improvements, maintenance and surveillance actions, or additional studies aimed at reducing uncertainty.

All proposed measures are prioritised through the DGA's GARI platform, which allows all recommendations to be managed consistently using the EWACSLs indicator, combining the principles of efficiency and equity in order to establish an implementation sequence for investments.

APPLICATION EXAMPLE: GUADALQUIVIR RIVER BASIN

As an example of the application of the methodology described above, this paper presents the study carried out in the Guadalquivir River Basin, which includes a total of 55 dams and two irrigation ponds with different typologies, ages and conditions.

This basin constitutes a representative example given the heterogeneity of the infrastructures analysed and their relevance in water resources management in southern Spain.

Failure modes identified and risk in the semi-quantitative phase

In the semi-quantitative phase, a total of 722 failure modes were identified for the set of dams and reservoirs analysed. These were classified into three scenarios according to the loads considered: hydrological, seismic and normal operation.

Figure 2 shows the number of failure modes identified for each dam and their distribution by scenario. Of the total, 297 were associated with dam breach scenarios, while the remaining 425 corresponded to non-breach failure modes.

Implementing a National Risk-Informed Framework for Dam Safety: Lessons Learned from Spain

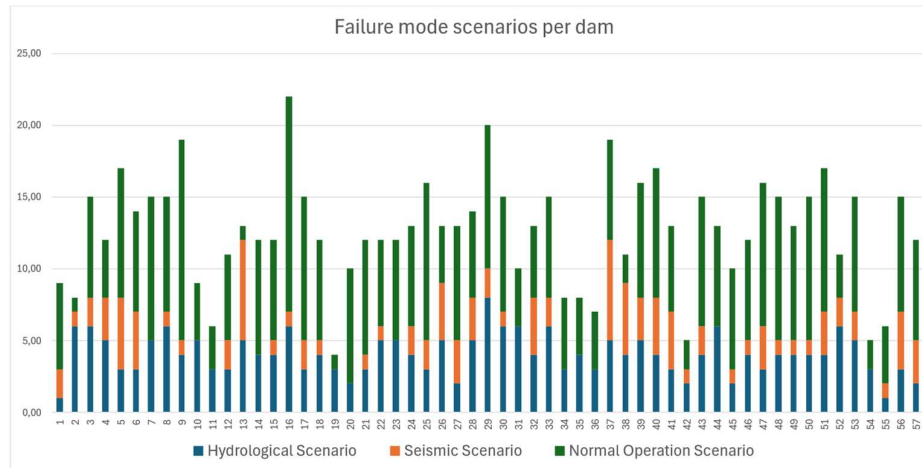


Figure 2. Number of failure modes by scenario and dam

Each failure mode was then classified in risk terms (low, medium or high) according to the probability of occurrence and the associated consequences, based on the assessment carried out by the multidisciplinary technical team. In total, 67 were classified as high risk, 367 as medium risk and 295 as low risk.

As a result of this phase, a semi-quantitative risk classification was obtained at dam level, as the average risk of its failure modes, as shown in Figure 3.

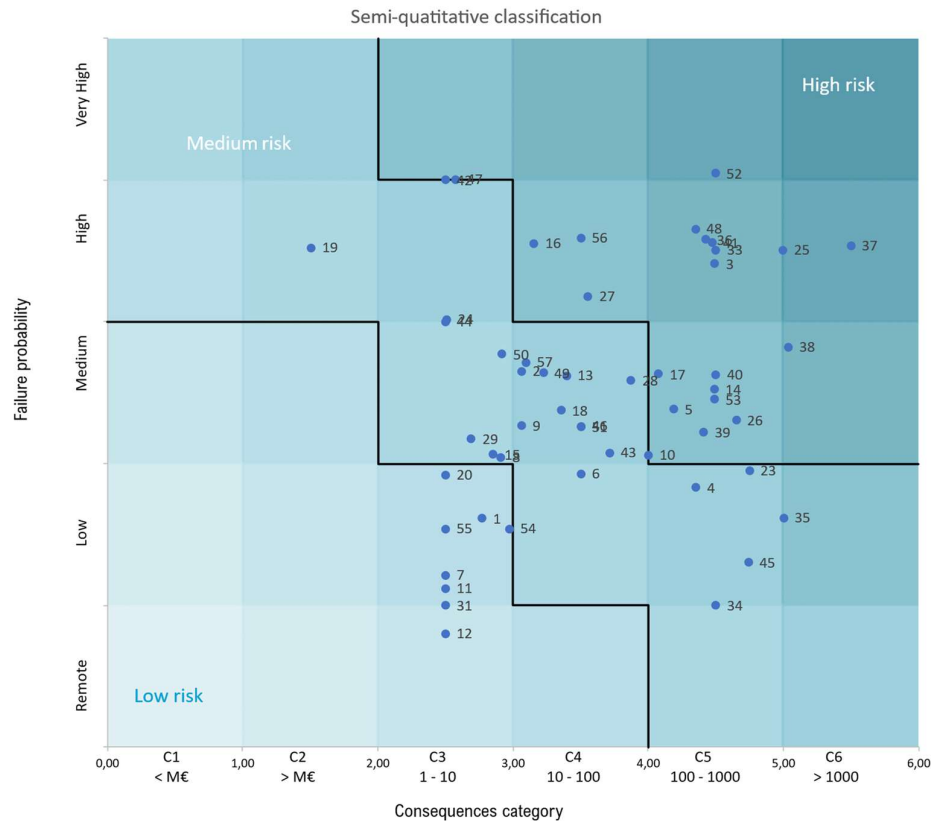


Figure 3. Average risk obtained for each dam analysed in the semi-quantitative phase

On average, the set of dams falls within a medium-risk level, with a probability of failure in the range of 10^{-5} to 10^{-4} and a C4 consequence category, corresponding to 10 to 100 potential fatalities.

These results make it possible to identify the dams with the highest risk levels in the basin and provide the basis for the subsequent quantitative analysis.

Quantitative risk results

Based on the failure modes identified in the previous phase, a quantitative risk model was developed for each of the dams and ponds in order to estimate failure probability and associated consequences numerically. In total, 77 failure modes were selected, ensuring that all dams were represented in the risk model by at least one failure mode.

As shown in Figure 4, most failure modes being analysed correspond to overtopping leading to dam breach. However, some geotechnical and seismic failure modes with high-risk levels in the semi-quantitative phase

Implementing a National Risk-Informed Framework for Dam Safety: Lessons Learned from Spain

could not be analysed at this stage due to the lack of information and studies required to reduce uncertainty in the models.

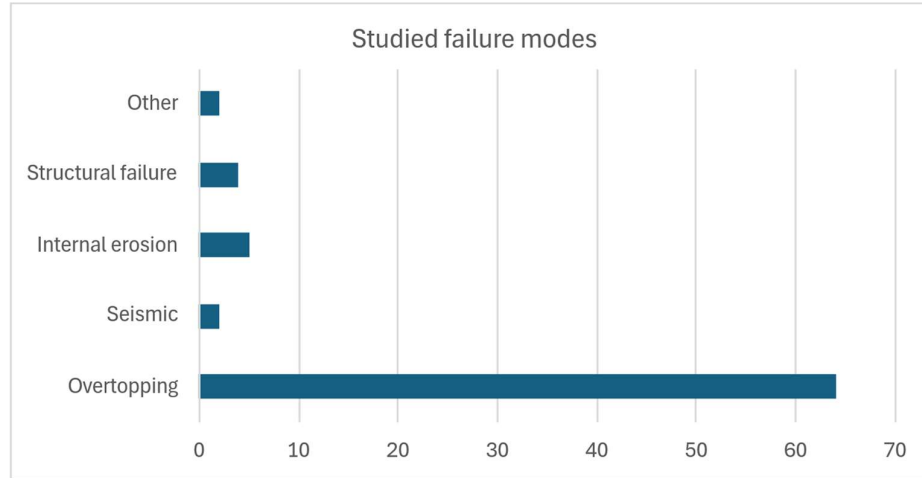


Figure 4. Failure modes included in the quantitative risk models

Results for the initial situation (2025)

Figure 5 presents the incremental risk results for the dams in the initial situation (2025), considering the average rates of change for the basin over the 2011–2040 period in the calculation of hydrological loads, as well as the estimated consequences based on the characteristics of the downstream areas.

The results show that 17 dams comply with the international social risk tolerability criteria, whereas 40 dams do not. In addition, 11 of the latter present particularly high-risk values, with potential losses exceeding 1000 lives.

These results clearly demonstrate the need to implement risk reduction measures.

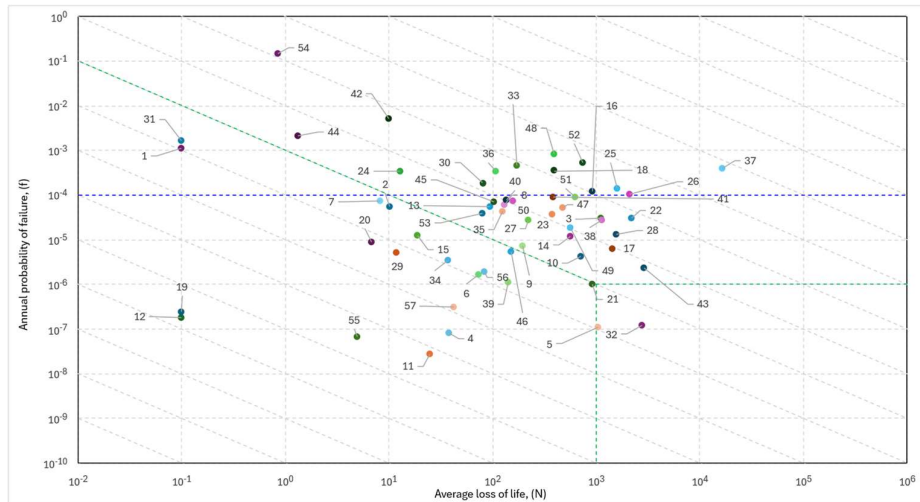


Figure 5. Incremental risk of the dams in the initial situation (2025)

Results for the medium-term situation (2055)

Figure 6 shows the results for the medium-term situation (2055), in which a general increase in risk can be observed compared with the initial situation. This increase is mainly associated with the rising intensity of flood events, which modifies hydrological loads, and with possible changes in downstream exposure.

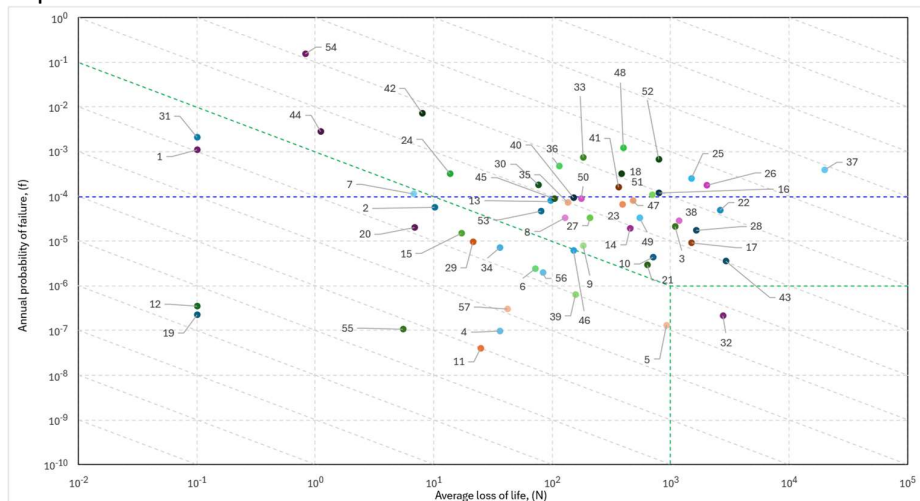


Figure 6. Incremental risk of the dams in the medium-term situation (2055)

These results underline the importance of incorporating climate change into risk assessment when planning future actions.

Implementing a National Risk-Informed Framework for Dam Safety: Lessons Learned from Spain

Effect of the measures and prioritization

Once the risk reduction measures had been defined, their effect on the results obtained was assessed and an implementation sequence was established in order to maximise their effectiveness in reducing risk.

Figure 7 shows the incremental risk results after implementation of the proposed measures in the 2025 scenario, while Figure 8 presents the reduction in social risk compared with the initial situation for the same year. As shown, after applying the full set of measures, 54 out of the 57 dams and ponds comply with the international social risk tolerability criteria. Of the three remaining, one does not comply with the social risk criterion and three do not comply with the individual risk criterion, presenting a high probability of failure but very low or incidental social consequences. In addition, three dams continue to present particularly high social consequences.

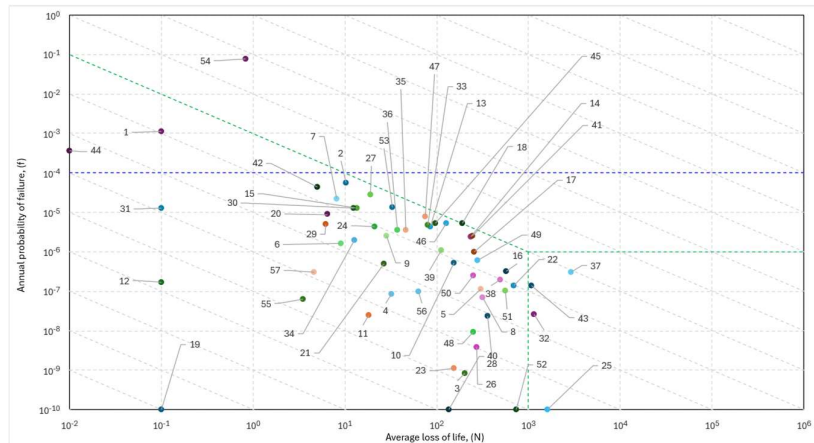


Figure 7. Incremental risk after implementation of the measures (2025)

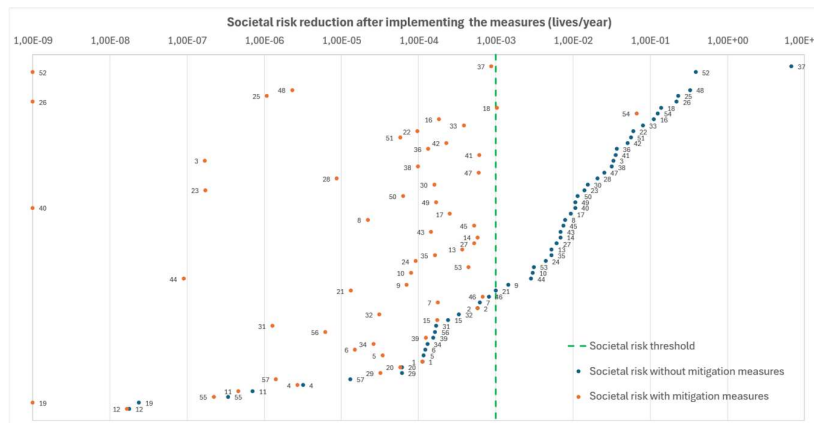


Figure 8. Social risk reduction after implementation of the measures (2025)

Figure 9 shows the resulting risk after implementation of the measures in the 2055 scenario. As in the previous case, the results show a significant improvement in the overall safety level of the dam portfolio.

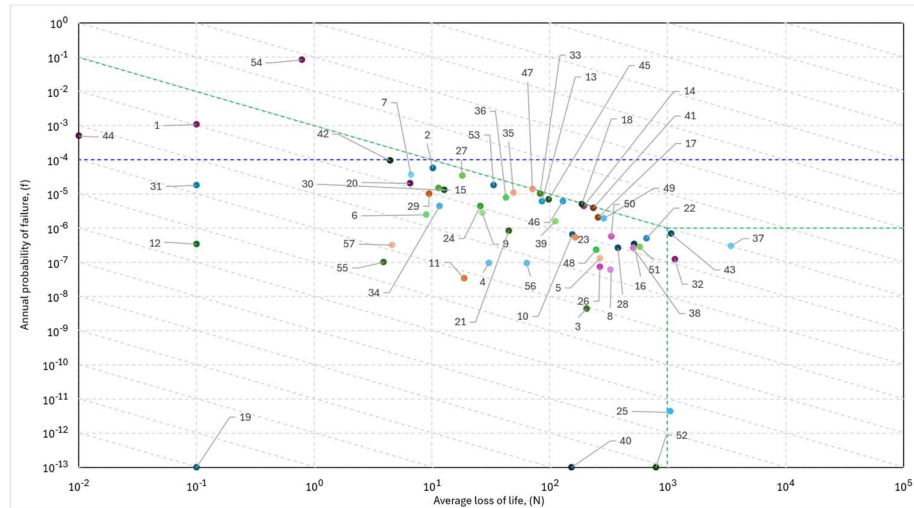


Figure 9. Incremental risk in the 2055 situation after implementation of the measures

Among all the measures analysed, certain structural solutions, such as parapet walls and dam heightening, stand out for their favourable effect on risk reduction, together with non-structural measures related to the implementation and/or strengthening of Emergency Action Plans. All of them have a highly significant impact on risk reduction, either by decreasing the probability of failure or by reducing the potential number of fatalities.

The total estimated cost of implementing the measures in the Guadalquivir River Basin amounts to EUR 511.60 million, which highlights the importance of establishing appropriate prioritisation criteria. The final implementation sequence was therefore defined by combining efficiency and equity criteria in resource allocation across the whole portfolio.

CONCLUSIONS

The implementation of risk analysis across the different river basin districts in Spain makes it possible to obtain a global, homogeneous and comparable view of the safety status of state-owned dams. This overview allows the most vulnerable infrastructures to be identified and, consequently, priorities for intervention to be established.

The application of the methodology to the Guadalquivir River Basin highlights the predominance of hydrological risk, as well as the effectiveness of combining structural and non-structural measures in reducing risk.

Implementing a National Risk-Informed Framework for Dam Safety: Lessons Learned from Spain

Actions of different nature contribute in a complementary manner to improvising safety, either by reducing the probability of failure or by mitigating its consequences.

The results also demonstrate the importance of incorporating climate change into this procedure, given the observed tendency for risk to increase under future scenarios. These results reinforce the need to adapt dam safety management to the changing conditions expected in the future.

Nevertheless, the implementation of the methodology has also revealed a number of challenges, including the availability and quality of information, the management of uncertainty associated with certain failure mechanisms, and the need to coordinate and harmonise criteria across different teams and river basin districts.

Overall, this methodology proves valuable as a tool to support strategic decision-making at national scale, improving dam safety governance in a more efficient and transparent way.

REFERENCES

- Spanish National Committee on Large Dams (SPANCOLD, 2012). *Dam Safety Technical Guide P-8: Risk Analysis Applied to Dam and Reservoir Safety Management*.
- A Morales Torres et al. (2026) Results of investment prioritization based on risk analysis in 300 Spanish state-owned dams. *Proceedings of the 94th ICOLD Annual Meeting*, Guadalajara, Mexico.
- USBR (2022). *Public Protection Guidelines: A Risk Informed Framework to Support Dam Safety Decision-Making*. Dam Safety Office. United States Bureau of Reclamation. United States Department of the Interior.
- USACE (2014). *Safety of Dams – Policy and Procedures*. Engineering and Design. United States Army Corps of Engineers. Washington, D.C.: United States Army Corps of Engineers.
- ANCOLD (2003). *Guidelines on Risk Assessment*. Australian National Committee on Large Dams



The British Dam Society

Benefits of Portfolio Risk assessment – a case study on flood detention reservoirs

A J BROWN, Jacobs
D BULLOCK, Binnies
A P COURTNADGE, Jacobs
I K KNIGHT, Jacobs
B PORRAS-CAMACHO, Binnies
C W SCOTT, Binnies

SYNOPSIS

The Environment Agency (EA) is carrying out a portfolio risk assessment of its ± 220 reservoirs, using the Tier 2 methodology given in the 'Guide to Risk Assessment for Reservoir Safety management (RARS)', (EA 2013) but modified to apply to flood detention reservoirs (FDRs) as described in Bullock and Gill (2026).

This paper describes the findings and draft results of a trial of the application of this Excel workbook tool on 20 FDRs.

The project includes data collection for the other 200 reservoirs owned by the EA, and this paper discusses the issues in achieving this.

Finally, it discusses how the output from a portfolio risk assessment (PRA) may be used to inform capital and operational spending using a risk-informed approach.

INTRODUCTION

The EA as undertaker for ± 220 FDR in England is implementing a Tier 2 PRA of their dams, to better prioritise reservoir safety risk assessment and management. The methodology, based on RARS, has been described in a paper by Bullock and Gill (2026), which built on a paper from the 2025 BDS prize competition. This earlier paper describes how the methodology in the 'Guide to Risk Assessment for Reservoir Safety Management' (RARS) was modified to allow application to FDR, as part of proactive management of the EA's portfolio.

A key difference from RARS is that for most of the EA reservoirs the availability of the dam break mapping carried out to the EA Reservoir Flood Mapping (RFM) specification (EA, 2024) enabled the use of more accurate data in assessing ASLL than the standard RARS approach.

Benefits of Portfolio Risk assessment – a case study on flood detention reservoirs

This paper describes application of the modified methodology in Bullock and Gill (2026) to a trial of 20 FDRs and presents a high-level summary of the draft key findings.

The project also includes collection of data on the remaining 200 reservoirs, to allow application of Tier 2 RARS as a follow-on project, with this paper describing the process for data collection, and some of the issues affecting quality.

OVERVIEW OF CHARACTERISTICS OF ENVIRONMENT AGENCY PORTFOLIO

Understanding the characteristics of an Undertaker's portfolio is important to ensure the risk assessment process is appropriate. Prior to development of the modified methodology, a review of the characteristics of the EA's portfolio was carried out.

A small proportion of the EA's portfolio are concrete dams which were not considered in this project. The portfolio of embankment FDRs comprises approximately 46% washland reservoirs and 54% impounding reservoirs. A summary of median values of EA portfolio physical characteristics compared to median UK dams is shown in *Table 1*. The data shows the typical EA FDR is much younger than and approximately half the height of a typical UK dam.

Table 1 Summary of median EA portfolio FDR vs UK median dam characteristics

	EA Portfolio	Washlands Only	Impounding Only	UK ¹
Dam height (m)	4	3	5	8
Year built	1985	1973	1994	1900
Reservoir Capacity (m³)	295,000	773,000	118,000	130,000

Step 3 of RARS focusses on proportionality, which is based on the value to prevent a fatality (VPF). Therefore, consideration of the potential consequences to human life if dams in the EA portfolio failed is important. A plot of the average societal life loss (ASLL) of washlands versus impounding EA FDRs based on RFM data is provided in Figure 1. The data shows that nearly 50% of washland reservoirs and over 30% of dry day failures of impounding reservoirs have ASLL less than 0.10.

As ASLL across the EA portfolio is generally low, sometimes proportionality is driven by property damage values rather than risk to human life. For

¹ Bowles, et al (2013) Median age from BRE Database, 1994. Note parameters may have changed since then as around 10 new dams have been built and 10 discontinued.

washlands, although 50% have property damage of £10M or more, around 10% have potential property damage of less than £0.1M.

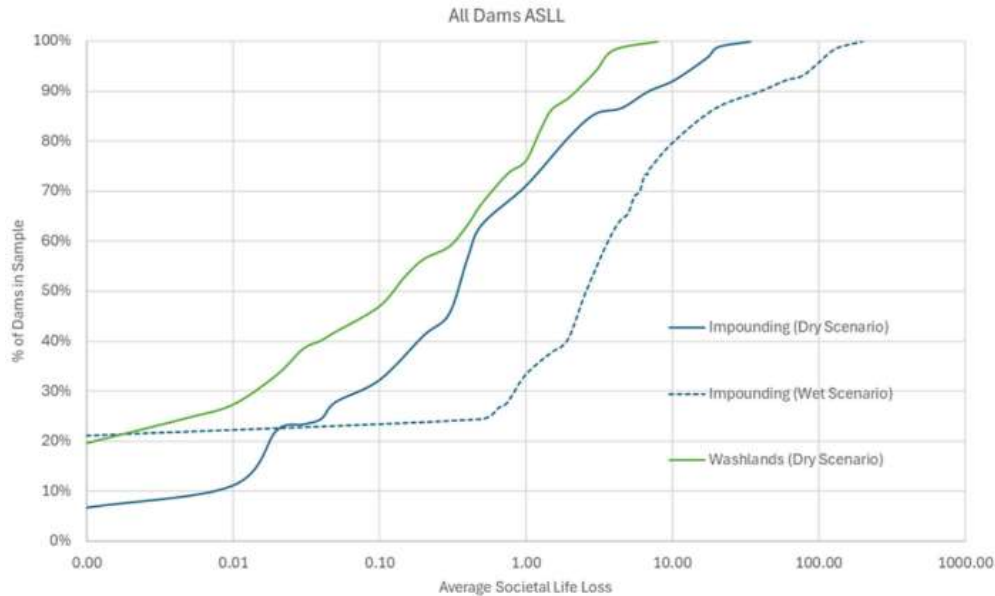


Figure 1 Plot of Average Societal Life Loss in EA portfolio of FDRs

EA portfolio data was taken from Tier 1 RARS analysis completed in 2021 to 2023 and some limitations in data for individual dams should be noted, but overall, the plots provide an overview of potential consequences of failure.

TRIAL ON REFINEMENTS TO RARS FOR FLOOD DETENTION RESERVOIRS

Process and dams selected for trial

Twenty reservoirs were selected for the trial, ten impounding and ten washlands FDR, split equally between Jacobs and Binnies. The selection was made to cover the range of features in the EA portfolio, including reservoir age, height, spillway arrangement including surface material, and flow control method including dams with large gates. The 20 dams were also drawn from across EA areas so the data-gathering process could be tested at the same time.

Each reservoir was assessed and checked internally by a consultant unfamiliar with it, then reviewed by an All Reservoirs Panel Engineer (ARPE) of the other consultant who is familiar with the site. This cross-consultant approach tests whether the workbook produces sensible results in unfamiliar hands, and if results appear reasonable to a reviewer familiar with the site. *Table 2* lists the 20 dams together with the assessor and reviewer for each. Following a preliminary trial in Autumn 2025 on 11 dams,

Benefits of Portfolio Risk assessment – a case study on flood detention reservoirs

refinements were made to the workbook. At the time of writing this paper, the assessments are being re-run on all 20 sites using an updated version of the workbook with this paper showing available results on the 20 sites.

Table 2 Trial dams and assessment team

No.	Assessor	Reviewer (ARPE)	Date built	Reason for selection
IR1	E1	ARPE1	1990s	Old reservoir, poor records
IR2	E2	ARPE2	2020s	Modern dam
IR3	E3	ARPE3	2000s	Concrete spillway, good records
IR4	E4	ARPE4	2010s	Homogeneous earthfill, known slope stability issues
IR5	E11 / E3	ARPE2	1990s	Gabion basket spillway
IR6	E3	ARPE1	2020s	High dam, Grasscrete spillway, no RFM
IR7	E5	ARPE1	1980s	Large gates
IR8	E6	ARPE4	1960s	Average grass spillway
IR9	E7	ARPE3	2000s	Earth embankment
IR10	E8	ARPE4	2000s	Modern dam, good records
W1	E8	ARPE4	2020s	Mesh spillway, passive management
W2	E9	ARPE3	2020s	Open mat spillway, active management
W3	E3	ARPE1	1980s	Typical washland
W4	E4	ARPE3	1150s	Old dam, grass spillway
W5	E4	ARPE4	1960s	Concrete inlet spillway
W6	E9	ARPE3	1660s	Old dam, grass spillway
W7	E9	ARPE1	1200s	Grass spillway, poor records
W8	E12	ARPE2	1950s	Typical Yorkshire washland
W9	E4	ARPE1	2000s	Excellent records, passive management
W10	E10	ARPE3	1960s	Grass spillway, active management

Results and findings – internal threats

Internal threat scoring reflected differences between washland and impounding FDRs. Impounding dam intrinsic-condition (IC) scores ranged between 2.7 and 8.0, with the higher end driven by older sites with limited records. Washland dams scored higher overall, between 5.3 and 9.0, reflecting the longer banks of mixed material and patchier construction history that are typical of these reservoirs. Current-condition (CC) scores generally tracked IC but were lower for impounding FDRs where surveillance is well established.

A consistent finding from the early assessments was that scores ran high where assessors defaulted to conservative inputs under uncertainty, lack of familiarity with RARS, or less experience identifying threats which would lead to failure. As noted in Bullock and Gill (2026), the assessor can modify scores ± 1 based on user judgement. This was used five times across three dams (four reductions and one increase). *Figure 2* shows the range of estimated probability of failure (PoF) associated with internal threats for the trial FDRs, which generally varied by three orders of magnitude. (absolute values anonymised)

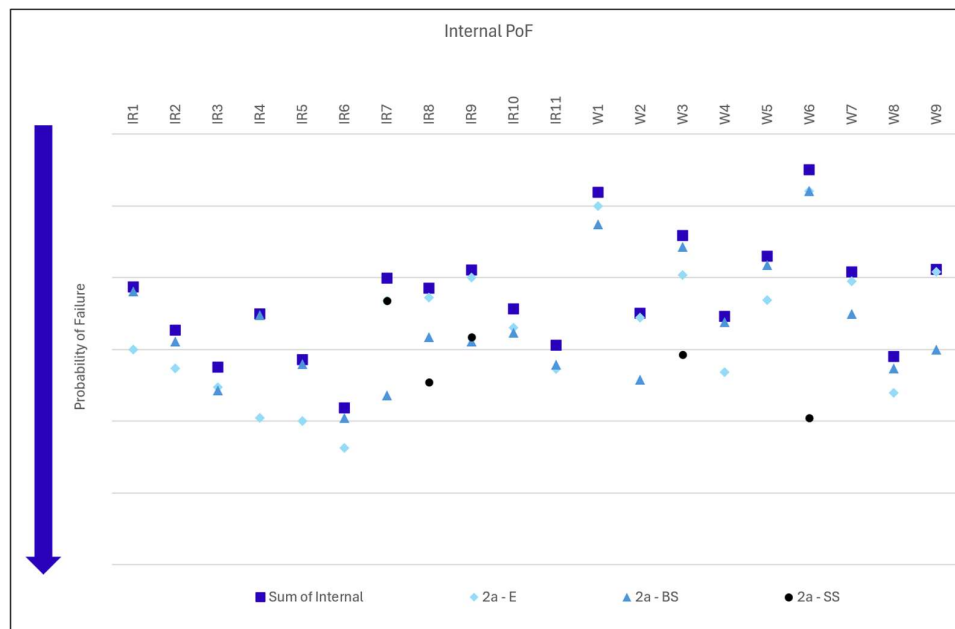


Figure 2 Annual probability of failure from internal threats

Results and findings – external threats

The overall probability of failure (PoF) due to overflow (Step 2b-1) is insignificant for most of the trial dams, often below 10^{-11} , because the

Benefits of Portfolio Risk assessment – a case study on flood detention reservoirs

structures are designed for no damage in dam safety floods, and the failure flood can be two to three times PMF.

PoF due to static slope instability (Step 2b-3) is the larger contributor in almost every case, with values typically low on modern dams and highest on the older reservoirs in the trial. The three highest-scoring dams are all 1960s reservoirs with limited construction records, which together drive the upper end of the range. Figure 3 shows external threat PoF, with slope stability (Step 2b-3) and overtopping (Step 2b-1) shown alongside the total so the underlying pattern can be seen.

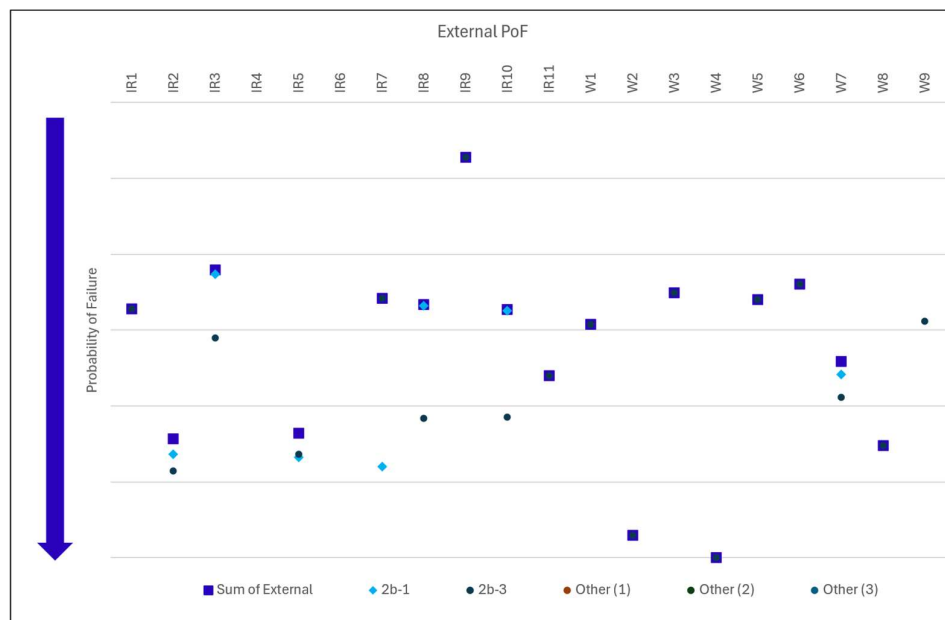


Figure 3 External PoF, including by failure mode

Results and findings – dam break and consequences

Consequence outputs span four orders of magnitude across the trial, with ASLL for the whole EA portfolio shown in Figure 1, being similar to the trial dams.

Results and findings – Risk of reservoir release

The Fatality vs. Number (FN) chart categorises the societal risk for the trial dams. The completed assessments generally fall within the Broadly Acceptable region or the ALARP Range.

Societal risk ($PoF \times N$) has a narrower spread than PoF alone, because the dams with the highest PoF have low N, and the dam with the highest N has only a modest PoF. For the dams furthest into the ALARP Range, the

dominant contributor is consistently a single failure mode rather than aggregated risk across several modes: slope stability for the early 1960s reservoirs (one impounding, two washlands), and internal threats on the buried structure for one washland. This is the key portfolio-management output.

A feature of the PRA for the EA was that the workbook also categorises risk to qualitative receptors shown in Tables 9.4 to 9.9 of RARS (EA, 2013), such as environment and cultural heritage. Results of risk to qualitative receptors for trial dams gave levels for impact typically one to two orders of magnitude higher than the impact on people and property, such that the RARS methodology suggests that these were the highest risk.

Step 3 Risk evaluation

As described in Bullock and Gill (2026) the economic analysis in Step 3c of RARS was reversed, to calculate the cost that would be proportionate to reduce the PoF by a factor of 2 (Scenario 1) and 10 (Scenario 2), with results shown in Figure 4.

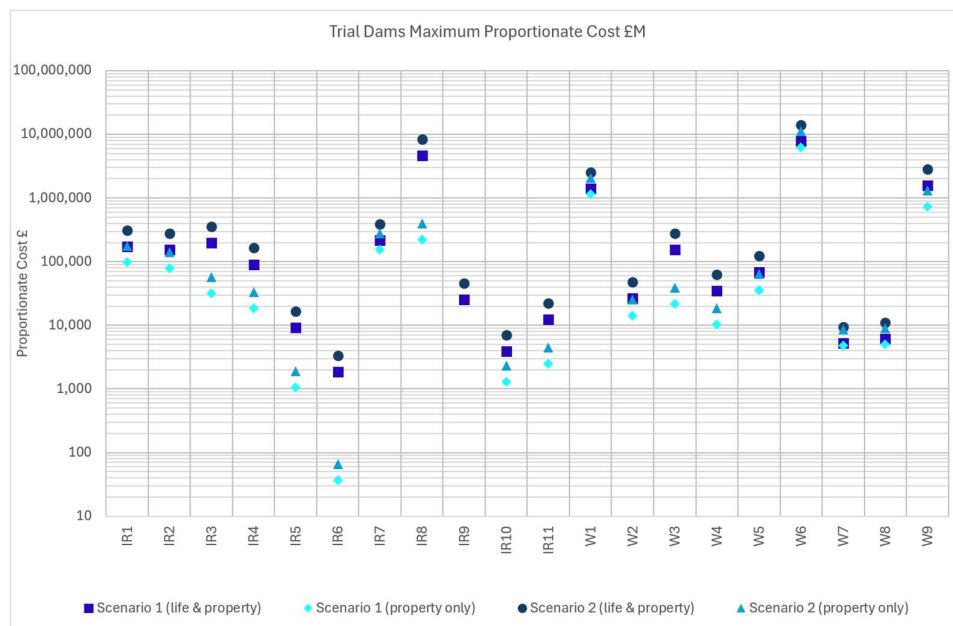


Figure 4 Maximum proportionate cost across the trial cohort (life & property and property only)

Discussion – accuracy of estimates of risk

The potential consequences of failure of the trial dams varies by four orders of magnitude, which is considered reasonable, and reflects a defined process for dambreak and consequences. The cumulative distribution of the trial

Benefits of Portfolio Risk assessment – a case study on flood detention reservoirs

dams plots between the “small dams” and dams in FD2641 shown on Figure 15.2 of RARS (EA, 2013).

The overall annual PoF varies by two to three orders of magnitude. FDRs are relatively recent so there are no long historical records of performance with which to compare these estimates. However, there is the following published distribution of overall PoF, which can be used as comparison

- a) Figure 15.3 of RARS which combines PRA from several owners and individual risk assessment on smaller dams (EA, 2013)
- b) The distribution from the trial of the risk assessment system which was formalised in the Interim Guide (Brown and Gosden, 2004)
- c) Recorded data on incidents and failures of UK dams in Brown and Tedd (2003)

There was some debate on whether FDRs would have a similar distribution of PoF to reservoirs which are normally full. On one hand, the chance of a FDR filling and loading the dam might be typically 1 in 100. On the other hand, many have never filled so they are in effect similar to new dams, when the PoF is much higher in the first five years after construction, which would offset the reduced annual chance of loading. The expected PoF due to internal erosion might therefore be similar to reservoirs which are normally full.

The overall distribution of PoF is similar to that of the available published distribution and it was concluded that the modified RARS methodology was reasonable and consistent with current good practice for other UK reservoirs.

Discussion – Ranking of dams

One of the key benefits of RARS and PRA is ranking the risk from the candidate dams, to define which are highest risk and the reasons for this, thus allowing investment to be prioritised accordingly. Some suggest the value of RARS is ranking, not necessarily absolute values which is more difficult to verify.

The three highest PoF dams in the trial are due to slope-stability problems, where the right response is investigation and slope works rather than spillway upgrades. The highest PoF washland is driven by the buried structures, where the proportionate response is surveillance and inspection. Without that fault-specific resolution, a high overall PoF could lead to uniform investment, which on this evidence could be a disproportionate call. The proportionate-cost output from the workbook, shown in Figure 4, gives the upper bound for justified spend on each dam.

DATA COLLECTION & REVIEW

Objective

The current project was to collect the data on the other 200 reservoirs to allow Tier 2 RARS to be carried out under future contracts. This needs sufficient good quality data to enable analysis of the reservoir's capacity to safely deal with external threats (primarily flood flows) and internal threats (construction and deterioration). The data quality should be proportionate to the potential consequences of failure.

Challenges and limitations

The first challenge was that although the initial objective was to collect all the data needed to complete a risk assessment, it was soon realised that this was more complicated than it first appeared. Most washland and many impounding FDRs are retained by multiple dams, and it is not always clear which is the highest risk dam or buried structures. Consideration was given to either collecting data on all the dams, or only the two highest risk dams, but the latter would have required interviews with the Supervising Engineer (SE) to better understand the risk posed by each dam and would have in effect been the first part of the risk assessment.

The project was then descoped to collect and catalogue available data, but not to identify the two highest risk dams or collect missing data. The screening of critical locations and the interview with the SE will be needed as an early part of the full Tier 2 RARS.

Process

Most of the reservoirs had Tier 1 RARS completed in 2021 to 2023 and that data had been retained on the consultants' project archives. A file list was provided to the EA to review against its records, with a request to supply any missing files and provide any files newer than January 2020. Some limitations on data handling and access were required due to files being password protected and/or classed as Official-Sensitive.

The key data required is set out in the EA data pack guidance (EA, 2021). The data review process is completed in an Excel worksheet tab as shown in *Table 3*, whilst the quality scoring system is shown in *Table 4*.

Benefits of Portfolio Risk assessment – a case study on flood detention reservoirs

Table 3 Tabs in data review workbook

Tab	Title	Purpose/Contents
1	incoming data register	
2	Reservoir “basics”	Swiftly understand the main site features, plus ASLL & damages from RFM flood modelling summary sheet.
3	High Level Review (HLR) of key data and documents	Understand the reservoir's layout, operation and main structures, to help inform the drawing review
4	Data (completeness & quality) Review & Scoring (DRS)	Within each row on each sheet, quality grades are assigned. The Grades align with AIMS scoring (EA, 2025) and reflect how much influence the data quality will have on the risk assessment.
5	Export table	Results are summarised for transfer to a portfolio-wide summary

Table 4 Data quality grades

Grade	Definition	Outcome from data review
1. Very Good	Complete, detailed, accurate, current & consistent data	Proceed with assessment
2. Good	Minor defects will not significantly affect the assessment	Proceed with assessment, noting minor uncertainties
3. Fair	Partial information, more could be simply obtained to improve the assessment	Proceed with assessment, with sensitivity testing of key uncertainties
4. Poor	Major defects would significantly affect the assessment	Proceed with assessment in parallel with seeking further information where proportionate*
5. Very Poor	Severe defects or lack of data making the assessment unreliable	Seeking further information where proportionate* before proceeding with assessment

* “proportionate” could be considered in terms of cost, effort, access, consents, programme, etc, in comparison to reduction in uncertainty and risk / consequences of failure.

Test for proportionate cost of investigations

At the start of the project, it was anticipated that there may need to be substantial data collection, in terms of topographic survey and ground investigation prior to carrying out Tier 2 RARS. However, after reviewing the range of characteristics of the reservoirs it was realised that washlands, being in the flood plain, often had low (or zero) risk to life. A test was therefore added for what cost would be proportionate for data collection, based on the ASLL given in RFM, and a typical annual PoF of about $1E^{-04}$. This saved the client significant sums in investigations for reservoirs where ASLL was 0.00 and deferred the decision on data collection until the Tier 2 RARS had been completed.

Lessons learnt

Experience from this project to collect, summarise and review quality of data on 200 reservoirs may be useful to other undertakers in developing portfolio-wide data reviews and assessments:

1. Filing systems differ between operational teams, so a standardised approach to data provision is helpful. A prompt list can be provided, which allows for data which does not exist to be identified rather than being shown as missing.
2. Data may be stored within several different parts of the Undertaker's organisation, such as reservoir safety, maintenance and capital works; so an enquiring mind and individual who can search all databases is invaluable, together with early population of the data register
3. A technique that was invaluable was comparing the number of drawings in the PFOR against the total number in other registers of drawings such as phases of construction works, with differences of over 100% in some cases
4. File contents are often not clear from filenames, and multiple revisions or out of date versions are supplied. This requires extra scrutiny to develop good data registers, which can then be used in updating the PFOR and other records.
5. Data extraction / review is not a "handle turning" exercise. Staff need to understand what to look for, where to find it, and critically whether it is useful. Experienced staff must be available to assist with key issues.
6. The risk of dam failure for FDRs is typically low, so data collection, review and assessment must be efficient and proportionate. Budgets for further studies or investigations will be limited and will need to be prioritised across a portfolio.

Benefits of Portfolio Risk assessment – a case study on flood detention reservoirs

DISCUSSION

The benefits of a portfolio risk assessment are primarily that it allows a reservoir owner to “step back” and review their group of reservoirs more strategically, in terms of factors such as risk management and operation.

For the EA portfolio the main benefits of the Tier 1 risk assessment had been (Brown et al, 2024):

- a) it defined reservoir safety floods for washlands and thus consistency in comparing the reservoir safety risk with impounding reservoirs (Courtnadge and Brown 2022)
- b) it identified that the definition of “consequence class” for environmental receptor was such that this often resulted in the highest (i.e. a class 4) consequence even when there is no population at risk

At Tier 2, RARS plots environmental receptors separately from property damage and life loss, but the highest “risk” as defined in RARS is often environmental receptors. The Tier 2 PRA confirmed this was an area where further consideration is required in definition of level of consequences.

A significant issue in the Tier 2 assessment was getting consistency between different assessment teams, and this led to recommendations of minimum grades of staff to carry out an assessment, and that review should be by an ARPE (Table 4 in Bullock and Gill, 2026).

Development of the tool, trial and associated review workshops:

- a) Allowed critical review of methods of dambreak and consequence assessment, as for the lower height dams the methodology in the RFM specification sometimes appears overconservative. Further work is undergoing to quantify this, and situations when the Undertaker may wish to carry out alternative more realistic analyses
- b) Identified that as well as risk to life and property, the need for improvements or upgrades to reservoirs may be driven by operational requirements, such as safe access for operators, including compliance with the owners “minimum technical requirements”.
- c) Produced a risk assessment which can be provided to ARPEs carrying out S10 inspections and/or acting as QCEs, to help assist in risk informed decision making.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the support of the Environment Agency for consent to publish this paper. They also gratefully acknowledge the

support of colleagues at Binnies and Jacobs in carrying out the project and writing the paper.

The opinions expressed in this paper are those of the authors, and do not necessarily reflect the views of the Environment Agency.

REFERENCES

- BRE (1994) *Register of British dams. BR 261*. Building Research Establishment, Watford, UK. ISBN 0 85125 630 9
- Brown A, Courtnadge A, Panzeri M, Goff C, Atyeo M, Coombs M and Davis A (2024). Risk assessment of existing flood detention (storage) reservoirs. In *Managing Risks for Dams and Reservoirs – Proceedings of the 2024 British Dam Society Conference*. British Dam Society, London, UK. See <https://britishdams.org/meetings-and-events/bds-conferences/conference-proceedings/>
- Brown A J and Gosden J D (2004) *Interim Guide to Quantitative Risk Assessment for UK Reservoirs*. Thomas Telford, London, UK (available on BDS website under Historical Guidance documents - <https://britishdams.org/reservoir-safety/res-safety-resources/#historical>)
- Brown A and Tedd P, (2003), The probability of a dam safety incident at an embankment dam, based on historical data, *Hydropower & Dams*. **10(2)**
- Bullock D and Gill C (2026) Developing a workbook and user guide to apply tier 2 RARS to EA flood detention reservoir. *Dams and Reservoirs* **36(2)** <https://doi.org/10.1680/jdare.26.00302>
- Courtnadge A and Brown A (2022) Washland flood detention reservoirs - Developing a consistent approach for assessing reservoir safety aspects. In *Dams and reservoirs in a Climate of Change – Proceedings of the 21st Biennial Conference of the British Dam Society*. British Dam Society, London, UK pp 219-231
- EA (2013) *Guide to Risk Assessment for Reservoir Safety Management. Volume 1: Guide (Report SC090001/R1). Volume 2 Methodology*. Environment Agency, Bristol, UK. www.gov.uk
- EA (2021) *Reservoir owner and operator guidance: inspection information pack*. Environment Agency, Bristol, UK. <https://www.gov.uk/guidance/reservoir-owner-and-operator-guidance-inspection-information-pack>

Benefits of Portfolio Risk assessment – a case study on flood detention reservoirs

EA (2024) *Reservoir Flood Mapping Specification. (Report ENVFCPMM00606B00C, v1.3)*. Environment Agency, Bristol, UK.
Available on request

EA (2025) *Flood risk asset maintenance and inspection: good practice guidance*. Environment Agency, Bristol, UK.
<https://www.gov.uk/government/publications/flood-risk-asset-maintenance-and-inspection-good-practice-guidance>



The British Dam Society

Testing Reservoir Emergency Arrangements: Lessons from a Post-Incident Exercise at Towcester

J CUBBAGE, Environment Agency

N HARDING, Reservoir & Water Services

SYNOPSIS

This paper describes how the Environment Agency reviewed, revised and tested the on-site emergency flood plan (OSEP) for Towcester Flood Storage Reservoir following a flood incident during Storm Bert in November 2024. The review identified limitations in operational assumptions and the plan's useability during a fast-paced incident. The OSEP was subsequently streamlined, removing generic content and refocused on the needs of trained incident response staff.

The revised plan was tested through Exercise Marmalade, comprising a desktop simulation and site-based activities to assess decision-making, communication and operational response under pressure. The exercise confirmed improvements in clarity and accessibility, while identifying further refinements around roles, training and operational readiness. The paper demonstrates that continuous improvement, supported by realistic testing, results in a concise, user-focused plan that enables effective emergency response.

INTRODUCTION

In November 2024, a flood incident at Towcester Flood Storage Reservoir during Storm Bert highlighted challenges in the Environment Agency's operational response and the practical use of the On-Site Emergency Flood Plan (OSEP). Although the incident was managed safely, it exposed some of the realities of acting under pressure.

Following an internal review, the OSEP was significantly revised to improve clarity, structure and suitability for trained incident response staff. However, this raised a key question: would these changes improve effectiveness in a real incident?

To test this, Exercise Marmalade was developed and delivered, combining a desktop simulation and site-based activities to assess decision-making and procedures under pressure. This paper outlines that process and demonstrates how focused document design and realistic testing can support more effective and reliable reservoir emergency response.

Testing Reservoir Emergency Arrangements: Lessons from a Post-Incident Exercise at Towcester

TOWCESTER FLOOD STORAGE RESERVOIR

Towcester Flood Storage Reservoir (FSR) is situated where the A5 (Watling Street) crosses the River Tove on the town's northern edge. The Tove drains a 105km² catchment.

Upstream of the A5, the river splits into the Tove and Mill Channel, passing under the road via two historic bridges. These restrict flood flows, forcing the river on to the floodplains until it spills over the A5 and into properties.

The impounding reservoir was constructed in 2002 and is owned, operated and maintained by the Environment Agency (EA). It comprises a combination of earth embankments, concrete floodwalls, steel floodgates, brick bridge parapets and two additional flood relief culverts under the A5 to provide flood protection up to the 2% AEP event.

TOWCESTER ON-SITE EMERGENCY FLOOD PLAN DEVELOPMENT

As reservoir undertaker, the Environment Agency updated the original on-site emergency flood plan (OSEP) in 2022 following the Ministerial Direction (Defra 2021). The new version followed a standard EA template designed to provide readers unfamiliar with FSRs and the EA incident management process with information needed to manage a reservoir safety incident. This included a substantial amount of peripheral material and generic management procedures.

It became clear that this sizeable document required improvement following an internal review of a flood incident which occurred during Storm Bert in November 2024.

IMPACT OF STORM BERT, NOVEMBER 2024

In November 2024, heavy rain in the Tove catchment caused river levels to reach the trigger to close the six reservoir gates as per standard operating procedures. Despite usually having more than six hours to travel to site and close the gates, the intensity of Storm Bert's rainfall meant water had already reached the gate openings when duty operations staff arrived.

They were unable to fully close the last pair of double gates due to the force of water through the opening. This led to 10 properties flooding due to water flowing down the A5. Duty staff were subsequently able to reduce the flow of water using plastic sheeting and sandbags until the water levels dropped a few hours later.

The Reservoir Supervising Engineer was informed throughout, and was satisfied that there was no safety risk to the reservoir structure.

REVIEW OF EA RESPONSE TO STORM BERT INCIDENT AT TOWCESTER

Following the incident, an internal review identified several lessons. Most notably, the rate of reservoir inflow exceeded the assumptions used when the gate closure trigger levels were originally established. The trigger level was subsequently lowered to allow more time for operatives to travel to site.

Another significant finding was that the OSEP contained a wide range of supporting information on EA incident management and Towcester generally, but this made key operational details difficult to locate quickly during an incident.

Using feedback from the duty incident staff and in collaboration with the Reservoir Supervising Engineer, the EA significantly amended the plan. Key information was reorganised; generic information removed and a clear focus kept on the target users: trained incident response staff. This reduced the document content by more than half. The intention was that these changes would allow easier navigation around the document and a brevity suited to emergency response situations.

The question now was: would it work any better? Fortunately, Towcester FSR had previously been programmed to be exercised in September 2025 giving us an opportunity to answer that question.

TESTING THE ON-SITE EMERGENCY FLOOD PLAN: EXERCISE MARMALADE

The exercise was split over two days: first using a desktop simulation, then testing emergency response activities on site. As Towcester FSR has no active outlet control structures, this element of the Direction would naturally be limited. However, it incorporated a test of the capability to deliver initial actions such as pumps and temporary defences.

It was identified that the most valuable feedback would come from staff who use the OSEP in an incident. This provided a dual benefit: identifying improvements to the OSEP, while also offering a safe training environment for staff to build familiarity with the document and practise incident response processes.

The Reservoir Supervising Engineer agreed with the plans and assisted and provided direction throughout the planning process.

Aims and Objectives

To provide a clear measure of success, aims and objectives were agreed with the Reservoir Supervising Engineer, including:

Testing Reservoir Emergency Arrangements: Lessons from a Post-Incident Exercise at Towcester

- Ensuring relevant EA staff understand the purpose of the OSEP, when to enact it, roles and responsibilities, and how it integrates with associated operational documents
- Stress-testing OSEP usability in a desktop simulation of an extreme rainfall event in the Towcester catchment
- Training EA operational field staff in a realistic environment to build familiarity with OSEP procedures and evaluate pump supplier response
- Using feedback from staff participating in the exercise to continuously improve the OSEP content and layout

Early discussions with the Reservoir Supervising Engineer focused the exercise on OSEP procedures, their links to EA Incident Management (EA IM) processes, and internal communication during the initial stages of a reservoir safety incident. EA IM duty staff manage a wide range of environmental and flooding incidents and operate 24/7, scaling up the response as events escalate. However, most are not reservoir specialists and reservoir safety incidents are not regularly exercised currently. The focus was therefore on testing the initial response to a significant safety event, where anticipated pinch-points included familiarity with the revised OSEP and timely communication and decision-making.

Exercise Marmalade Day 1: Desktop Simulation

Exercise Scenario

The storyboard of the exercise scenario used the November 2024 Storm Bert rainfall data to create the initial conditions and extended this event to simulate a worsening situation. The scenario was based around a deteriorating leak being discovered in the embankment. This would be found as part of the routine patrol of the embankment and walls carried out by the EA duty operational staff attending site when the reservoir impounds.

Exercise set-up

The exercise involved over 25 participants who were grouped into their EA IM roles and based on separate tables:

- Flood Incident Duty Officer (FIDO) – leads all flood risk incident operations activities within the catchment, providing direction to FODO and escalating issues to ABC
- Field Operations Duty Officer (FODO) – manages the EA resources attending site

Cubbage and Harding

- Area Base Controller (ABC) – manages the overall response to the reservoir incident
- Area Duty Manager (ADM) – has overall responsibility for Incident Management in the Area

For the initial stages, the ABCs and ADMs were sent out of the room and provided with no further information. This reflects reality in the initial stages of an incident where these roles rely on accurate and timely information being passed on by FIDO and FODO in line with the OSEP procedures. At that point, they were brought back into the exercise room. This was felt to be important based on previous experiences of EA IM desktop exercises where early involvement of ABCs and ADMs often led to an unrealistic level of direction during the initial stages of an incident.

Exercise execution

On receiving each inject for their specific role, participants used information and procedures in the OSEP to evaluate the issue, take actions, make decisions and escalate as required. They replicated phone calls by moving between tables to communicate with other roles, allowing a test of information flow and accuracy, and whether OSEP processes were followed.

In addition to carrying out their roles and completing duty logs, participants were asked questions with each inject. This enabled direct evaluation of OSEP usability: how easily information was found, how it related to associated documents, and how well it aligned with wider incident management processes. Answering the questions also had the benefit of increasing participants' familiarity with the updated OSEP.

The responsibility for completing these tasks rotated within each role group, ensuring everyone was actively involved and maximising individuals' learning opportunity.

At the end of each inject, the exercise paused to review actions and responses to the questions. This allowed facilitators to confirm the scenario remained on track and correct any deviations so that subsequent injects made sense. It also provided participants with short breaks during what proved to be quite a high-energy, fast-paced session.

Exercise feedback

From previous experience of desktop exercises, it was recognised that participants are not always aware of how they are using documents during a scenario. An observer was therefore assigned to each group to capture how the OSEP was used in practice, identifying omissions or areas for

Testing Reservoir Emergency Arrangements: Lessons from a Post-Incident Exercise at Towcester

improvement. Their observations provided valuable feedback that would otherwise have been missed.

In addition to this, further information was collected, during and after the desktop exercise, to enable a review the effectiveness of both the OSEP and the simulation itself:

- Points raised during the discussion session after each inject
- Feedback form about the use of the OSEP
- Feedback form about the effectiveness of the exercise
- Written responses to the inject questions
- Completed duty logs from each role

Exercise Marmalade Day 2: On Site Test and Training

As a flood storage reservoir, the area remains inactive for long periods before becoming fully impounded. Most of the operational staff therefore have limited real-life experience of managing incidents when they occur. The site-based element of Exercise Marmalade focused on training EA operational staff while testing sourcing and supply of pumps for drawdown. Pumps were brought to site and set up in locations identified in operational pump plans.

OUTCOMES FROM EXERCISE MARMALADE

Exercise evidence and feedback from both days was used to identify refinements to the OSEP document as well as review the EA incident management response to a reservoir safety event at Towcester. Ultimately, the aim was to establish whether the changes made to the OSEP after the November 2024 incident had genuinely improved emergency response under pressure.

Improvements to the On-Site Emergency Plan

Feedback on the OSEP was positive and participants of the desktop simulation confirmed they found the revised plan easy to find and straightforward to navigate. Although there were no significant limitations found with the revised OSEP, improvements were suggested for some of the decision-making stages.

The desktop simulation also highlighted the need to create a single, easily accessible location for all reservoir safety documents

Lessons Identified in Incident Management Response

The desktop simulation demonstrated that clearly defined roles are required to manage a high volume of information exchanges. In a fast-paced, evolving situation, there is limited time to assess, make and communicate decisions, whilst maintaining accurate records throughout. The simulation also highlighted the need for more familiarisation training with reservoir safety incidents generally.

The site activities on day 2 highlighted several opportunities for improvement in the IM arrangements and procedures:

- The lead time required for the supply of high volume pumps presented challenges due to the location of the reservoir in the upper part of the flashy River Tove catchment.
- An aide memoir was recommended to assist operations staff when gathering information during the reconnaissance phase of a reservoir incident.
- The exercise strongly reinforced the need for development and training of the Field Operations Site Controller role, providing leadership to operations field staff on site and a single point of contact for EA activity during an incident. Exercise Marmalade demonstrated a clear need for further investment in this role, locally and nationally.
- EA duty incident staff are encouraged to 'Think Big, Act Early, Be Visible' and this exercise underlined the need to deploy additional resources to site in the early stages of a potential incident.

Desktop simulation execution feedback

Participants felt the structure and delivery of the desktop simulation worked well for both OSEP familiarisation and general reservoir safety incident training.

Feedback emphasised the simulated 'phone calls' between tables being particularly effective, demonstrating how information was communicated, and highlighting bottlenecks in its flow.

The pause between injects to reflect on learning was welcomed, and the additional tasks with each inject were well received, enhancing learning value and helping to maintain engagement.

Testing Reservoir Emergency Arrangements: Lessons from a Post-Incident Exercise at Towcester

HOW TO MAXIMISE THE EFFECTIVENESS OF A RESERVOIR ON-SITE PLAN TEST?

The experience of the exercise showed that:

- Setting clear aims and tangible objectives, agreed with the Reservoir Supervising Engineer, provides direction, defines the intended outcomes, and identifies expected limitations. These objectives also help prioritise and inform decisions in the planning stage, such as how to run the exercise and what to include. In turn, this makes evaluating the success of the exercise more straightforward.
- While OSEP tests primarily focus on the performance of plans and procedures, they also provide a valuable opportunity to train staff about reservoir legislation, common failure modes, and what actions to take in a reservoir safety incident.
- Ensuring a test reflects realistic operating conditions is key to maximising its learning benefits.
- A large budget is not necessarily required to robustly test the OSEP.
- Continuous improvement is iterative; an incident should act as both the catalyst and a valuable input for reviewing the OSEP. Taking time to reflect on such events enables targeted enhancements to the document's content, structure and usability.

CONCLUSION

This paper describes how an incident at Towcester Flood Storage Reservoir was used as a catalyst to review, amend and subsequently test the On-Site emergency Plan. It sets out how clear objectives and priorities ensured the exercise focused on driving continuous improvement rather than simply verifying that the plan functioned adequately.

Using a desktop simulation to stress-test procedures, alongside initial on-site contingency actions, Exercise Marmalade provided evidence to confirm that post-incident changes to the Plan strengthened emergency response under pressure. Further refinements were identified, both to the On-Site Plan document and more broadly to the wider EA incident management approach. Exercise Marmalade delivered additional benefits including staff training and improved familiarity with reservoir safety legislation and procedures.

The paper concludes by highlighting key considerations for designing and delivering exercises that meaningfully test an On-Site Plan, including the

Cubbage and Harding

importance of clear objectives, realistic scenarios and active participant engagement.

Through the iterative review of the Towcester On-Site Plan, it has been demonstrated that a concise, well-designed OSEP plays a key role in the effective and efficient response to a reservoir safety incident. Realistic, robust testing helps identify information gaps and procedural pinch points. This enables targeted improvements to be made well before the rain starts to fall.

REFERENCES

Department for the Environment, Food and Rural Affairs (2021) *Flood Plan (Reservoirs Emergency Planning) Direction*, available at: www.gov.uk



The British Dam Society

Climate Change Impacts on Flood Risks – What Can Dam Owners Do?

J STUART, Climate Resilience PTY Ltd.

S PERERA, Murray Darling Basin Authority.

M HUSSEIN, Stantec UK Ltd.

L KING, Klohn Crippen Berger.

SYNOPSIS

Climate change is altering the hydro-meteorological processes that govern extreme floods, presenting new challenges for dam safety assessment, spillway design, reservoir operations and governance. While the physical basis for intensification of extreme precipitation is well established, substantial uncertainty remains regarding how flood frequency distributions will evolve in individual dam catchments given spatial, temporal and topographical factors. At the same time, dam safety frameworks depend on historical data, the use of which relies on a stationary climate and stable hydrological conditions. Recent global dam incidents have heightened awareness of these risks, yet regulatory guidance remains fragmented and inconsistent.

Drawing on the recent ICOLD Technical Committee Y Bulletin on Flood Risk Evolution Associated with Climate Change, this paper translates emerging science into practical guidance for dam practitioners within the United Kingdom (UK) risk-informed framework. The paper summarises key physical drivers of flood change, discusses the implications of a non-stationarity climate for flood estimation, examines the limitations of climate models for extreme event projection; and proposes a structured and iterative adaptive management framework to facilitate risk-informed prioritisation guidance on where to focus resources for portfolio dam owners.

A process is presented that proposes application of a six-step pathway that combines top-down ensemble/multi-model analysis with bottom-up stress testing, relevant to high-consequence dams. The paper argues that the appropriate response to climate uncertainty is not deterministic prediction or blanket design inflation, but structured, risk-informed, adaptive decision-making supported by enhanced monitoring, vulnerability analysis and clarity of risk appetite. The underlying principle is that dam owners should describe uncertainty rather than attempt to eliminate it and should enhance or retain safety margins where it can be done economically. This approach aligns with

Climate Change Impacts on Flood Risks – What Can Dam Owners Do?

a risk-informed philosophy and supports proportionate, defensible decision making under significant uncertainty.

INTRODUCTION

Flood loading remains one of the principal contributors to dam incidents and failures worldwide. ICOLD Bulletin 188 (ICOLD, 2020) identifies inadequate spillway capacity as a leading cause of failure, with the proportion of hydrologically driven incidents increasing since 2000. Meteorological and hydrological events have been identified as the root causes for about 25 percent of failures of large dams in the 21st century, though these events often interact with other vulnerabilities such as under-design, maintenance neglect and gate malfunction (Liu et al., 2025; Nasser et al., 2025).

Historically, flood estimation methods have relied on the assumption of hydrological stationarity; the idea that climate variability fluctuates within a stable envelope. This assumption is now untenable. Milly et al. (2008) states that *“stationarity is dead and should no longer serve as a central, default assumption in water resource risk assessment and planning”* and the ICOLD Technical Committee Y Bulletin on Flood Risk Evolution Associated with Climate Change (ICOLD, 2026) reinforces this conclusion, noting that climate change introduces significant uncertainties associated with estimates of shifting flood frequency curves. Arguably, we are moving into a different phase of management: ‘weather change’ that perhaps highlights the bespoke nature of the problem and the speed with which it has advanced in the last quarter century.

For dam practitioners, the primary discussion relates to practical activity, with net zero an important but secondary consideration. The discussion now centres on how to incorporate the evolving climate into dam safety decisions, manage significant uncertainty in flood estimation and remain proportionate and defensible under legal/regulatory scrutiny.

This paper responds to these challenges by outlining key physical drivers of changing flood behaviour, reviewing approaches for estimating non-stationary floods, examining uncertainty implications and presenting a practical framework consistent with best practice.

PHYSICAL DRIVERS OF FLOOD RISK

Climate change influences many aspects of the hydrological cycle in complex ways. The overall flood response varies from catchment to catchment, depending on basin characteristics, storm patterns, land use conditions and large-scale climate oscillations. ICOLD (2026) notes that it is difficult to provide single, definitive statements that apply universally, highlighting the importance of local assessment. Also, these drivers can occur together and

strengthen one another; and their overall impact on flood frequency at any location depends heavily on local conditions. There has always been and will remain a wide variety of rainfall events impacting catchments globally. However, the change is occurring both vertically; what might be called atmospheric change in existing conditions, and also spatially where catchments that were once exposed to a temperate climate, for example, may, in future be exposed to sub-tropical rainfall. The local and global nature of the matters at hand demonstrate the complexity of the situation the sector faces.

Intensification of Extreme Precipitation

One of the broadly agreed features of a warming climate is that extreme precipitation is expected to intensify. The Clausius–Clapeyron relationship shows that the atmosphere can hold about 7% more moisture for every degree Celsius of warming, and observations confirm that short duration rainfall extremes often increase at a similar rate (Fowler et al., 2021). In some areas, especially for convective storms, even higher “super Clausius–Clapeyron” scaling has been recorded (Wasko et al., 2021). However, the ICOLD 2026 bulletin notes that this thermodynamic effect interacts with complex atmospheric dynamics, such as storm tracks and large-scale circulation patterns (IPCC, 2021). This means that higher rainfall intensity does not automatically lead to more frequent events, nor does it produce uniform changes across all catchments or climate zones.

For UK dam practitioners, a broader uncertainty envelope results with extreme events in Norfolk (2020) and Suffolk (2021) akin to sub-tropical climates demonstrating the need to consider not just the rainfall, but what is driving rainfall. In the case of the Suffolk event, post event analysis Holley et al, (2022) suggests the 1-hour (92mm) precipitation record for the UK were exceeded by 45% when informal data is considered. Similar events have rewritten preparedness and response approaches in France, when the Pas de Calais region saw historically significant and extended rainfall totals at multiple stations in November 2023 (Meteo France, 2023) beyond the design of a range of civil infrastructure and response planning considerations.

With intensity, frequency and duration continuing to evolve, regular reassessment becomes a necessity, particularly where response times are short and mechanism for future rainfalls may differ significantly from existing design estimation methods. The atmospheric rivers that impacted the UK in 2015 (Storm Desmond) and in 2024 (Storm Bert) (Figure 1) demonstrate the ability for rainfall with sub-tropical intensities and origins to occur in the UK. Warming ocean temperatures and greater atmospheric

Climate Change Impacts on Flood Risks – What Can Dam Owners Do?

moisture seem only likely to increase this risk with information available at the time of writing, and it is correct to say ‘increase’ in the UK context; such flood events have been an intermittent feature of UK flood history. Rainfall in Norfolk, 1912 and Dorset in 1955 (275 mm in 9 hours) (Stirling, R, 1982) confirm this.

Sea Surface Temperatures

Elevated sea surface temperatures have been assessed as contributing as much as 40% to the rainfall volumes during the Valencian floods in 2024 (Ramiro et, al, 2026), and are a known driver of the ‘Medican’ that impacted substantial areas of the eastern Mediterranean and Libya in 2023. (Nemnen, 2025)

The ICOLD (2024) statement on the Derna and Abu Mansour dam failures in Libya reinforces how extreme hydrological events during cyclonic events combined with ageing infrastructure can lead to catastrophic outcomes. A post event investigation (Nemnen, 2025) suggests that region’s dams, even if structurally sound, would have provided minimal protection against such extreme runoff. For dam owners, this means that historical flood records may underestimate future risk and catchments previously considered low risk may require reassessment.

Atmospheric Rivers

Storm Desmond in 2015 demonstrated the impacts atmospheric rivers can have on the UK with extensive impacts noted in the north of England and a new 24-hour rainfall record. These features can link with rainfall drivers and provide an extended source of elevated moisture; again challenging existing design assumptions for water infrastructure Atmospheric rivers have long been a hazard for the UK, but with a warming atmosphere able to hold greater volumes as noted above, the changing risk is clear and is highlighted by the 2024 flooding caused by Storm Bert (Figure 1) that challenged design assumptions for flood risk response and infrastructure according to the post event review.

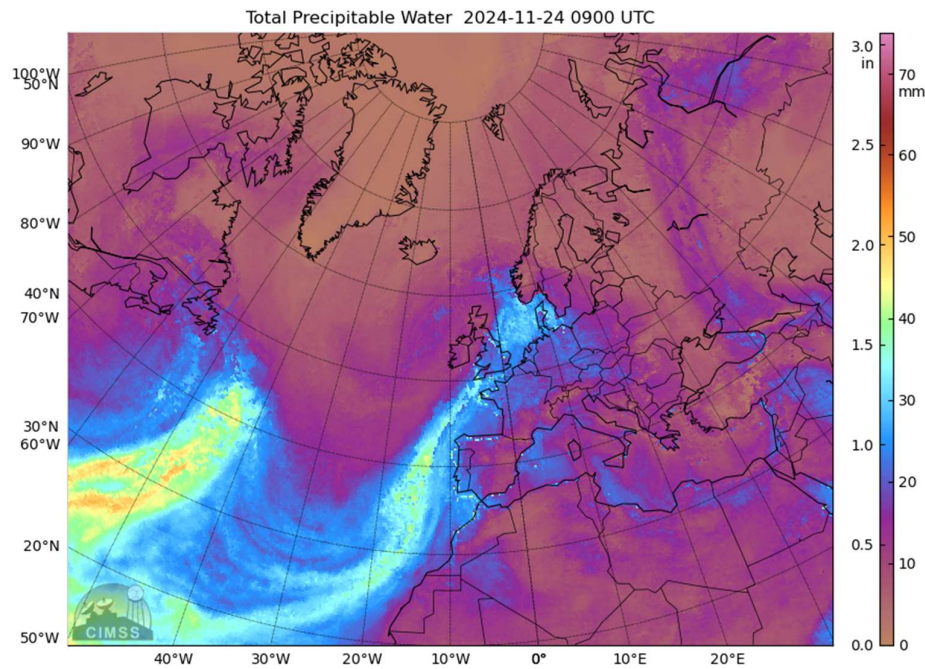


Figure 1: Storm Bert – Atmospheric River, 2024

Large Scale Climate Oscillations

Large scale oscillations such as the El Niño–Southern Oscillation (ENSO), the Indian Ocean Dipole and the North Atlantic Oscillation exert a strong influence over regional flood regimes. Climate change may alter the frequency, intensity or persistence of these oscillations, thereby modifying flood risk in ways that are difficult to predict (Cai et al., 2014). The ICOLD 2026 bulletin emphasises that these oscillations can amplify or mask long term climate change signals, contributing to multi-decadal variability in flood behaviour. For dam practitioners, this underscores the importance of long observational records, multi-member/multi-model ensemble approaches and caution when interpreting short term trends. However, such importance is also problematic moving forward. The transition underway means that practitioners must now consider significant uncertainty in areas that have traditionally provided anchor points in standards or risk-based decision making.

Land Use and Compound Hazards

Climate driven changes in land use and catchment condition including bushfire, vegetation loss, soil degradation and increased landslide activity can significantly alter runoff generation. Burned or degraded catchments often exhibit higher runoff coefficients, reduced infiltration and increased

Climate Change Impacts on Flood Risks – What Can Dam Owners Do?

debris flow potential. Compound events such as extreme rainfall occurring shortly after bushfire or during saturated soil conditions can produce nonlinear amplification of flood peaks (Nyman et al., 2015; Perera & Jolly, 2024). The ICOLD 2026 bulletin stresses that such compound hazards are becoming more common and must be explicitly considered in dam safety assessments.

Snowpack, Rain-on-Snow and Frost

In snow-dominated catchments, climate change is altering both the magnitude and timing of runoff generation. Rising temperatures can reduce seasonal snow accumulation, accelerate snowmelt, and shift precipitation from snowfall to rainfall. These changes may increase the frequency and severity of rain-on-snow and frost events, where rainfall occurs over an existing snowpack or frozen ground, causing rapid melt or increasing runoff; substantially increasing runoff volumes. The combined effects can lead to higher flood peaks, reduced predictability of inflow patterns, and shifts in flood seasonality. The extent of these impacts varies according to basin characteristics, including elevation, climate regime, and catchment response mechanisms. For dam owners and operators, these changes introduce additional uncertainty into flood hydrology assessments and may require reassessment of design floods, reservoir operating rules, and flood management strategies. The 1947 floods in the UK are notable for unusually high runoff after an extended period of freezing conditions (Stirling, R, 1982) causing some of the most severe floods of the 20th Century that included infrastructure failures and over 100,000 properties flooded.

NON-STATIONARITY FLOOD ESTIMATION

Traditional flood estimation methods rely on the assumption that hydrological processes fluctuate within a stable statistical envelope. Climate change fundamentally alters the reliability of this assumption. As noted above, the ICOLD 2026 bulletin notes that while the physical basis for increased atmospheric moisture is clear, the translation of this signal into flood frequency changes is highly uncertain and varies across regions and catchment types (Wasko et al., 2024). Some basins may experience upward shifts in the entire flood frequency distribution, while others may see changes concentrated in the upper tail (King & Micovic, 2023). For Probable Maximum Flood (PMF) estimation, the challenge is even greater: Probable Maximum Precipitation (PMP) methods assume that historical storm mechanisms remain valid, yet climate change may alter storm dynamics, moisture availability and storm tracks. As a result, deterministic adjustments to PMP or PMF values are not currently supported by consensus science; and sensitivity and scenario testing may provide the greatest value.

Climate change means historical flood records can no longer be assumed to represent future conditions. ANCOLD (2025) now expects design flood estimation to account for a non-stationary climate.

In the UK context, UKCP18 projections indicate an increase in rainfall intensity and the frequency of extreme events, particularly for short-duration convective storms. Consequently, design rainfall can no longer be considered temporally stationary and should instead be treated as a non-stationary, time-dependent variable. Environment Agency analyses indicate that approximately 13%–21% of UK gauging stations exhibit statistically significant positive trends in peak river flows¹, providing empirical evidence of an evolving flood regime. In practice, rather than replacing the Flood Estimation Handbook (FEH) framework, current UK approaches incorporate climate change allowances alongside sensitivity and scenario-based assessments to address this non-stationarity.

Because climate change allowances depend on the chosen warming level and planning horizon, scenario selection becomes central. In this evolving field, scenarios are best used to test system sensitivity across plausible futures rather than to support confidence in any single projection. In addition, current methods that apply a uniform uplift factor across design rainfalls inherently include an element of stationary climate behaviour and should be tested against climate zone predictions.

Non-stationarity has practical consequences for dam owners. The AEP of a given flow is no longer fixed. There has always been a significant envelope of uncertainty, and this is now changing as the climate evolves. A design flood that was once a 1-in-1,000 return period event may correspond to a more, or less frequent recurrence under future conditions.

Such changes may affect spillway adequacy, freeboard design, gate operation, emergency planning and the residual life of ageing assets. It also has implications for risk assessments. Incorporating climate change risk into major infrastructure decisions presents a significant challenge for dam owners, as such decisions must be based on projections that remain highly uncertain and subject to revision as modelling approaches and scientific understanding continue to evolve.

¹ [Development of interim national guidance on non-stationary fluvial flood frequency estimation - summary](#)

Climate Change Impacts on Flood Risks – What Can Dam Owners Do?

LIMITATIONS OF CLIMATE MODELS FOR EXTREMES

Global Climate Models (GCMs)

GCMs remain the primary tool for projecting future climate conditions, but their limitations are particularly acute for extreme precipitation. The ICOLD 2026 bulletin highlights that while temperature projections are robust across models, precipitation projections, especially for extremes, exhibit substantial divergence (IPCC, 2021, IPCC 2022). This is due to coarse spatial resolution, simplified representation of convective processes and structural differences between models. Despite these limitations, GCMs provide useful insights into the range of possible future conditions. For dam practitioners, the key is not to treat GCM outputs as forecasts or predictions, but as projections that can be used as inputs for stress testing and scenario exploration. It is considered best practice to consider an ensemble of multiple GCMs to explore uncertainty in projections between differing models.

Downscaling

Downscaling methods attempt to translate coarse GCM outputs into catchment scale information. Statistical downscaling methods such as quantile mapping can adjust biases in GCM precipitation distributions, but they rely on the assumption that statistical relationships remain valid under future climates; an assumption that may not hold for extremes. Dynamical downscaling using regional climate models provides physically based simulations at finer resolution but introduces additional model uncertainty and computational cost (Kay et al., 2021). Convection-permitting models and kilometre-scale GCMs can capture extreme rainfall more realistically. However, owing in part to significant computational demands, their use is still constrained by limited spatial coverage and relatively small ensembles. These models are expected to become more widely available over time but are also subject to some limitations such as topography, for example. The ICOLD 2026 bulletin emphasises that downscaling does not eliminate uncertainty; rather, it adds new layers of uncertainty that must be transparently communicated to decision-makers and relevant stakeholders.

Scenario Uncertainty

Future emissions trajectories diverge significantly after mid-century, leading to wide differences in projected warming and hydrological impacts. The ICOLD 2026 bulletin stresses the importance of using multiple scenarios (e.g., SSP2-4.5 and SSP5-8.5) to capture emissions scenario uncertainty (IPCC, 2021). Practitioners should avoid over-interpreting differences between scenarios, particularly for extreme rainfall. Instead, scenario

analysis should be used to explore the sensitivity of dam systems to a range of plausible future conditions.

UNCERTAINTY IN CLIMATE-DRIVEN FLOOD ASSESSMENT

Uncertainty in climate-informed flood estimation arises from multiple sources – some quantifiable, some non-quantifiable. Aleatory uncertainty reflects natural climate variability; the inherent randomness of rainfall and floods. It cannot be removed by better data. Epistemic uncertainty stems from limitations in scientific knowledge, including model structure, parameter setting and data quality. These categories of uncertainty are present in stationary flood estimates and attempts can be made to quantify them. Including non-stationarity in flood assessments adds additional uncertainty. Assessments rely on an even longer chain of models, with uncertainty being increased with each step in the modelling chain – GCM ensemble, emissions scenarios, downscaling methods, hydrological model setup, and choice of statistical distribution, for example. These sources of uncertainty are quantifiable and can produce wide uncertainty envelopes for future flood estimates (King & Micovic, 2023). On top of that are non-quantifiable sources of uncertainty relating to incomplete knowledge of the climate system and how the future will unfold. This substantial uncertainty has been referred to by some as “deep” uncertainty.

Effective communication of uncertainty is essential to avoid over-confidence and to support risk-informed decision making. The ICOLD 2026 bulletin provides a useful conceptual illustration showing how future uncertainty envelopes can exceed present day uncertainty, particularly for rare events.

The era of weather change?

A key area of emerging uncertainty is the ability of current weather models to accurately predict seasonal and daily weather. This was noted by the Australian Bureau of Meteorology in 2024 who advised that the skill in forecasts was declining (BOM, 2024). For dam owners, this highlights the need to have emergency response plans that consider a wide range of outcomes and uncertainty with plans tested regularly. In common law countries, it is also necessary to ensure risks are managed *So Far As Is Reasonably Practicable* (SFAIRP).

A STRUCTURED DECISION PATHWAY FOR DAM PRACTITIONERS

Adaptive Management Implementation Pathway

In light of the substantial uncertainties and limitations of the current state of the science, the 2026 bulletin recommends that dam owners adopt an adaptive management approach to addressing climate change impacts on

Climate Change Impacts on Flood Risks – What Can Dam Owners Do?

flood risk. Adaptive management decision pathways support proportionate and defensible climate-related dam safety decisions, where assessments are periodically updated and planning adjusted.

The approach is presented in *Figure 2* and in Table 1, along with how the approach aligns with ANCOLD guidance.



Figure 2 : Adaptive Management Framework

Table 1: Adaptive Management Framework Steps

Step	Activity	Key Questions	Typical Output	UK Alignment
1. Identify climate sensitive flood drivers	Screen climate drivers relevant to the catchment – both over acute and chronic timeframes.	Which drivers (extreme rainfall, snowmelt, rain-on-snow, SST, land-use) matter for this catchment?	Driver shortlist; data needs register.	Periodic flood study review normally instructed by Inspecting Engineer under Section 10 inspection.

Step	Activity	Key Questions	Typical Output	UK Alignment
2. Check current margins	Assess existing hydraulic, structural and operational margins.	Spillway capacity, freeboard, outlet works, gate reliability, operating rule flexibility: what margin exists today?	Margin map versus existing design and safety check floods.	Under Flood study review, freeboard assessment should be carried out following Floods and Reservoir Safety (ICE Guide) - 4th Edition.
3. Top-down enveloping	Develop a plausible envelope of future flood loads.	What range of design floods is plausible under current generation of GCMs and emissions scenarios?	Ensemble flood-frequency / PMF envelope.	Design flood assessment under UKCP18, FEH and EA/SEPA climate allowances, current reservoir safety practice defines the Probable Maximum Flood (PMF) based on present-day meteorological conditions and does not formally include a climate change uplift.
4. Bottom-up stress testing	Test system performance under incrementally increasing loads.	At what level of additional flood loading does a failure mode become credible? At what point do existing operations plans fail?	Vulnerability thresholds; failure mode triggers.	Robustness testing beyond current design compliance, sensitivity checks with spillway blockage scenarios proposed by Inspecting Engineer
5. Assess adaptation options	Evaluate operational, non-structural and structural options.	What options are available to mitigate risks? What is the most cost-effective sequence: operational, non-structural, structural?	Adaptation pathway with staged decision points.	Proportionate risk reduction through reservoir safety measures, drawdown capacity review, reduced TWL or revising operational rules,

Climate Change Impacts on Flood Risks – What Can Dam Owners Do?

Step	Activity	Key Questions	Typical Output	UK Alignment
6. Monitoring and re-analysis triggers	Establish indicators and triggers for re-evaluation.	Which observed indicators would prompt re-evaluation, and how often?	Monitoring plan; re-analysis triggers; Plan Do Check Act cycle.	Reappraisal triggers from inspections, events and new guidance.

Prior to moving through the adaptive management framework steps, it is recognised that a nuanced, systems-based understanding of dam resilience is required when considering climate change impacts on dams. Understanding the dam system components, their interactions and operational context is key to identifying potential vulnerabilities and assessing how climate change could impact these. It is important to develop an understanding of how hydrologic loading interacts with structural condition, mechanical reliability, sedimentation and operational response capability (ICOLD, 2020). For example, mechanical failure of gates during extreme events has been a contributing factor in many historical dam incidents. Accessibility to operate gates is another key aspect. This holistic understanding of the dam system provides the context for the adaptive management process. Additional details of the six-step pathway are as follows:

Step 1 — Identify climate sensitive flood drivers

This step corresponds to **hazard identification** under Section 10 of Reservoir Act 1975. A routine inspection is required after 10 years, periodic catchment-specific review of climate-sensitive hazards and available hydrological evidence is recommended.

Step 2 — Check current margins

This step aligns with **risk analysis** under Floods and Reservoir Safety (ICE Guide) - 4th Edition (2015), margin checks reveal how close the dam currently sits to critical thresholds (spillway capacity, freeboard, gate reliability). This establishes the baseline risk profile and identifies where climate change may push the system into intolerable risk zones.

Step 3 — Top-down enveloping

This step supports **likelihood analysis** under non-stationary conditions. By developing an envelope of plausible future floods considering UKCP18, FEH-based hydrology and applicable EA/SEPA climate change allowances to test plausible future flood loading and reservoir safety consequences under emissions scenarios, dam owners avoid over confidence in any single projection.

Step 4 — Bottom-up stress testing

This step maps directly to **failure mode analysis and risk evaluation**. Stress testing systematically varies flood loading to determine the points at which specific failure modes become credible. This step undertakes sensitivity / exceedance testing to demonstrate robustness of the reservoir system, not just compliance with current design assumptions.

Step 5 — Assess adaptation options

This step corresponds to **risk treatment**. Adaptation options are identified, and then evaluated in terms of robustness, cost-effectiveness, and sequencing. Consider proportionate risk-reduction measures through the normal reservoir safety process, e.g. spillway improvement, crest raising, enhanced drawdown capacity, operational changes and emergency planning.

Step 6 — Monitoring and re-analysis triggers

This step aligns with **monitoring and review**, reservoir supervision and periodic dam safety inspection under Reservoir Act 1975. By defining key inputs to monitor (e.g. rainfall trends, catchment condition, spillway performance) and identifying triggers for re-analysis (e.g. updated models and methodologies, planned dam upgrades), dam owners can embed climate change into ongoing dam safety governance. This also supports reappraisal triggers to reservoir category, inspection findings, significant hydrological events, updated climate guidance, and periodic flood study review.

Practical Considerations:

- The process presented above should consider a dam's consequence category to determine the level of analysis required. The level of effort can be tailored according to the consequence classification.
- Exposure to climate drivers such as extreme precipitation, snowpack changes or tropical cyclones must be assessed in the context of local catchment characteristics.
- Where warranted, top-down ensemble assessments can provide insight into potential future changes, while complimentary bottom-up stress testing can identify system thresholds and vulnerabilities.
- Margins of safety should be evaluated holistically following a systems approach that considers hydrological, structural, mechanical, and operational factors.
- Assumptions, uncertainties, and decisions should be transparently documented to support regulatory review and future reassessment.

Climate Change Impacts on Flood Risks – What Can Dam Owners Do?

Key Principles

The structured decision pathway highlights two key underpinning principles:

1. Characterise rather than eliminate uncertainty

Transparency in assumptions, data limitations, and uncertainty. Presenting projections as ranges and disclosing methodological choices aligns defensible, evidence-based decision making.

2. Prefer robust and adaptive options under uncertain conditions

The ALARP/SFAIRP principle and risk-informed philosophy favour options that perform reliably across a wide range of future conditions. This aligns with ICOLD (2016) and World Bank (2021) guidance on emphasising robustness and adaptive management strategies and avoiding hasty decisions.

AVOIDING TWO EXTREMES

The ICOLD 2026 bulletin cautions against two common but opposing pitfalls: complacency and overreaction. Complacency arises when practitioners assume that historical hydrological data remain fully valid despite clear evidence of climate change and fail to include it in future planning. Overreaction and alarmism, on the other hand, can lead to unnecessary expenditure and potential misallocation of resources.

A balanced, risk-informed approach grounded in uncertainty analysis, stress testing, and adaptive management provides a more defensible and effective pathway towards managing climate change risk in the face of substantial uncertainty. This involves developing a solid understanding of the dam system and implementing a long-term management strategy that involves periodic re-evaluation of impacts and re-assessment of response strategies as models, methodologies and at-site conditions evolve. Climate change does not necessarily mean universal enlargement of spillways. Enhancing operational flexibility, improving gate reliability and strengthening emergency management plans may provide greater risk reduction than structural upgrades alone.

RECOMMENDATIONS FOR DAM PRACTITIONERS

Six recommendations follow from the discussion above, consistent with international guidance.

- Drop the stationarity assumption. Treat unadjusted historical flood records only as a baseline. Apply Wasko et al. (2024) for design-flood estimation.

- Characterise rather than reduce uncertainty. Report design floods as ranges informed by GCM ensembles and multiple emissions scenarios, as well as multiple methodologies, where possible.
- Develop a solid understanding of the dam systems, their components and interactions to identify vulnerabilities and prioritise climate-sensitive hazards for analysis.
- Apply the six-step pathway in proportion to dam failure consequence. Use light-touch screening for low-consequence dams. Use the full top-down and bottom-up analysis for High and Extreme consequence dams.
- Sequence adaptation by considering reversibility and cost. Prioritise operational and non-structural measures first, with structural upgrades introduced as staged commitments linked to monitoring triggers.
- Embed climate-sensitive monitoring indicators with explicit re-analysis triggers (e.g. five-yearly review, new IPCC assessment cycle or planned dam upgrades).
- Strengthen organisational capability by building in-house literacy in climate science, ensemble methods and uncertainty communication. Collaborate across portfolios to share data and methods.

CONCLUSIONS

Climate change is altering the hydro-meteorological conditions in which dams operate. While substantial uncertainty remains, particularly in precipitation projections, the physical basis for more intense extreme rainfall is well established. The ICOLD 2026 bulletin emphasises that maintaining dam resilience is essential and that adaptive management must remain central to good practice.

For dam practitioners:

- Perfect prediction is unobtainable;
- Robustness and adaptive capacity are achievable.

Adaptive management provides a structured, iterative strategy to manage climate risks that is both practical and defensible. The future of dam safety under climate change will depend less on deterministic precision and more on structured, transparent and adaptive decision making. In this context, the decision pathway outlined in this paper offers a scalable framework for sequencing actions, embedding monitoring triggers, and ensuring that dam safety programs remain responsive as new climate information emerges.

Climate Change Impacts on Flood Risks – What Can Dam Owners Do?

ACKNOWLEDGEMENTS

The authors wish to thank fellow members of ICOLD Technical Committee Y on Climate Change, Dams and Reservoirs, in particular the chapter authors of the bulletin on Flood Risk Evolution Associated with Climate Change.

REFERENCES

- ANCOLD (2025). *Guidelines on Flood Capacity for Dams*. Australian National Committee on Large Dams.
- BOM, (2024). *Climate Driver Update, 16th April 2024*. Bureau of Meteorology
- Cai W et al, (2014). Increasing frequency of extreme El Niño events due to greenhouse warming. *Nature Climate Change*, **4** pp.111-116.
- Fowler H J et al (2021). Intensification of short-duration rainfall extremes and implications for flood risk: current state of the art and future directions. *Philosophical Transactions of the Royal Society A: Mathematical, Physical and Engineering Sciences*, **379**(2195), 20190541.
- Holley D et al. (2023). The Brettenham storm of 25 July 2021. *Weather*, **78**: 196-201. <https://doi.org/10.1002/wea.4145>
- ICOLD (2016). *Bulletin 169: Global Climate Change, Dams, Reservoirs and Related Water Resources*. International Commission on Large Dams, Paris, France
- ICOLD (2020). *Bulletin 188: Statistical analysis of dam failures*. International Commission on Large Dams, Paris, France
- ICOLD (2024). *Statement on the Derna Dams Failure (Libya, September 2023)*. International Commission on Large Dams, Paris, France.
- ICOLD (2026). *Flood Risk Evolution Associated with Climate Change*. ICOLD Technical Committee Y on Climate Change, Dams and Reservoirs. International Commission on Large Dams, Paris, France
- IPCC (2021). *Sixth Assessment Report, Working Group I: The Physical Science Basis*. Intergovernmental Panel on Climate Change. Cambridge University Press.
- IPCC (2022). Technical Summary. *Climate Change 2022: Impacts, Adaptation and Vulnerability. Contribution of Working Group II to the Sixth Assessment Report of the Intergovernmental Panel on Climate Change*. Cambridge University Press.
- Kay A et al (2021). Climate change impacts on peak river flows: Combining national-scale hydrological modelling and probabilistic projections. *Climate Risk Management*, **31**, 100263.

- King L M and Micovic Z (2023). A practical approach to characterizing climate change uncertainties on dam hydrologic loading. *Symposium "Management for Safe Dams" - 91st Annual ICOLD Meeting*.
- Liu L et al (2025). The Analysis and Warnings of Recent International Dam Failure Disasters. *Proceedings of the International ICOLD-CIBG Symposium "Common Challenges, Shared Future, Better Dams"*, held in May 2025 in Chengdu, China, pp. 78-91.
- Meteo France press release, 18th November 2023. Meteo France.
- Milly P CD et al (2008). Stationarity is dead: Whither water management? *Science* **319**: 573-574.
- Nasser R et al (2025) Statistical Analysis of Global Dam Accidents in the 21st Century: A Focus on Common Features and Causes. *Water Resources Management*, **39**, 6493–6516.
- Nemnen A et al (2025) How extreme rainfall and failing dams unleashed the Derna flood disaster. *Nature Journal*, 2025
- Nyman P et al (2015). Predicting sediment delivery from debris flows after wildfire, *Geomorphology*, **250**, Pages 173-186.
- Perera S and Jolly C (2024). *Impacts of Climate Change on Dams and Reservoirs*, ICOLD Symposium, Delhi, India.
- Ramiro I et al (2026) The key role of Mediterranean and North Atlantic sea surface temperatures on the 2024 record-breaking Valencia precipitation event, *Weather and Climate Extremes*, **52**, 2026, 100877, <https://doi.org/10.1016/j.wace.2026.100877>.
- Stirling R (1982) *The Weather of Britain*. Fakenham Presse Ltd. ISBN 0-571-11695-7
- Wasko C et al (2021). Evidence of shorter, more extreme rainfalls and increased flood variability under climate change. *Journal of Hydrology*, **603** (PB), 126994.
- Wasko C et al (2024). A systematic review of climate change science relevant to Australian design flood estimation. *Hydrology and Earth System Sciences*, **28(5)**, pp.1251-1285.
- World Bank (2021). *Resilient Water Infrastructure Design Brief*. World Bank Group, Washington DC.



The British Dam Society

Inspections of Small Raised Flood Storage Reservoirs

P D DOWN, Mott MacDonald
J GREENWAY, Environment Agency
J R P SYKES, Mott MacDonald

SYNOPSIS

The Reservoirs Act 1975 defined a 'large raised reservoir' as being *"...designed to hold, or capable of holding, 25,000 cubic metres of water..."* above the lowest natural level of the surrounding ground. The Flood and Water Management Act 2010 proposed reducing the threshold to 10,000m³. In England, this has not yet been implemented and the volume threshold remains unchanged. As part of Reservoir Safety Reforms being considered, it is proposed to implement legislation that will introduce the reduced threshold.

The Environment Agency (EA) is responsible for a stock of flood storage reservoirs, including those considered 'small raised reservoirs' with storage volumes between 10,000m³ to 25,000m³. In preparation for a reduction in the volume threshold and the need for 'small raised reservoirs' to then be operated and maintained in accordance with legislative requirements, inspections 'in the spirit of the Reservoirs Act 1975 (as amended)' were progressed. The aim was to identify significant issues that may need resolving and progress measures to address these where necessary.

This paper details the inspections for two example cases. It summarises the findings of the inspections performed and identifies issues that may be relevant to owners of other 'small raised reservoirs'.

INTRODUCTION

Legislation Background

Following two fatal dam failures in 1925, the Reservoirs (Safety Provisions) Act 1930 established a threshold for classifying a 'large reservoir' at 5 million gallons (approximately 22,700m³). The Reservoirs Act 1975 introduced legislative improvements, including supervision and enforcement, and increased the 'large raised reservoir' threshold to 25,000m³ (5.5 million gallons).

The Flood and Water Management Act 2010 amended the Reservoirs Act 1975 and introduced a reduced storage volume threshold of 10,000m³. The aim was to further improve public safety and risk management based on evidence that smaller reservoirs (that had been unregulated to date) could

Inspections of Small Raised Flood Storage Reservoirs

still pose significant risks to life should they fail (UK Parliament, 2017) or result in serious ‘near-miss’ incidents (Griffiths, 2012). However, the publication of Statutory Instrument 2013, No.1590 (HM Government, 2013) retained a storage volume threshold of 25,000m³ in England. The reduced threshold of 10,000m³ was implemented in Wales during 2016.

Following the Toddbrook reservoir incident in 2019, the findings of the independent reviews (Balmforth, 2020 & 2021) recommended a systematic review and development of reservoir safety legislation and the associated regulations and guidance. This resulted in a Reservoir Safety Reform programme (Environment Agency, 2026) launched and progressed in England and Wales. The vision was to create a safety regime for reservoir dams in England and Wales which protects communities. The proposed reforms include the reducing the storage volume threshold to 10,000m³ in England so that ‘small raised reservoirs’ will be subject to the same legislative requirements as ‘large raised reservoirs’.

Proactive Approach

Inspections ‘in the spirit of the Reservoirs Act 1975 (as amended)’ were progressed by the EA for the ‘small raised reservoirs’ for which it is the Undertaker. The aim was to identify any significant issues and progress measures to address them ahead of any legislative requirement to do so.

Terms and Clarifications

The following terms and clarifications apply to this paper:

- The Reservoirs Act 1975 (as amended) is, hereafter, referred to as ‘the Act’.
- ‘Undertaker’ is used for both contexts of an operator of a ‘large raised reservoir’ under the Act and of a ‘small raised reservoir’ being treated ‘in the spirit of’ the Act.
- ‘Flood Storage Basin’ (FSB) is used to differentiate the ‘small raised reservoir’ examples within this paper from a large raised ‘Flood Storage Reservoir’ (FSR). It was also the term used by the Undertaker for the case examples.

SCOPE OF WORKS

Mott MacDonald (MM), the Consultant, was commissioned to complete inspections of three ‘small raised reservoirs’ in the North-West of England, two in the Cumbria & Lancashire (CLA) area and one in the Greater Manchester, Merseyside & Cheshire (GMC) area. These inspections were commissioned as part of the EA National Reservoir Risk Identification & Reduction Programme (RRIRP) as a proactive approach in preparation for

upcoming legislative change. The aim of this programme was to identify risks associated with the Undertaker's stock of 'small raised reservoirs', with over forty assets being identified nationally.

The original inspection scope communicated in mid-October 2025 was for All Reservoir Panel Engineers to complete pseudo-Section 10 Inspections 'in the spirit of the Act' to identify potential Measures in the Interest of Safety, Maintenance Measures and other activities. The aim was to improve management of the assets in advance of legislative change. There was a requirement to have all inspection deliverables issued by end March 2026. A review of existing data was completed and all available information, including as-built records, modelling, etc. associated with the 'small raised reservoirs' provided for the inspections. Where necessary, appropriate topographical surveys were commissioned, completed and issued within a relatively short timeframe to inform the inspections. The inspections were completed on-site in late January 2026 with reporting provided to the Undertaker in March 2026.

The inspection findings for two of the reservoirs are detailed in this paper as example cases.

INSPECTION APPROACH

On receipt of the Undertaker's request, the Consultant identified there was limited availability of the All Reservoirs Panel Engineers within the specified timescale. As the 'small raised reservoirs' did not fall under the ambit of the Act, it was proposed that an experienced Supervising Engineer, already appointed by the Undertaker, perform the services and that these be overseen by designated All Reservoirs Panel Engineers. This approach was agreed.

STOCK BECK FSB

Stock Beck Flood Storage Basin (FSB) is located within a residential area to the east side of Kendal, Cumbria. It was constructed for the Local Authority. This was supervised 'in the spirit of the Act' by an All Reservoirs Panel Engineer who issued a Preliminary Certificate in June 2007, on substantial completion, and a Final Certificate in July 2011. Ownership and responsibility for the reservoir was transferred to the Environment Agency before December 2015.

The reservoir is an online impounding type and provides a flood storage function by controlling flows along the watercourse and storing excess water within the valley upstream of the embankment dam (Figure 1). It is indicated to have a total storage volume of 32,600m³ at a Top Water Level of

Inspections of Small Raised Flood Storage Reservoirs

52.33m AOD. However, some of this is below the surrounding ground level. The storage volume above the lowest adjacent ground level is 21,107m³.

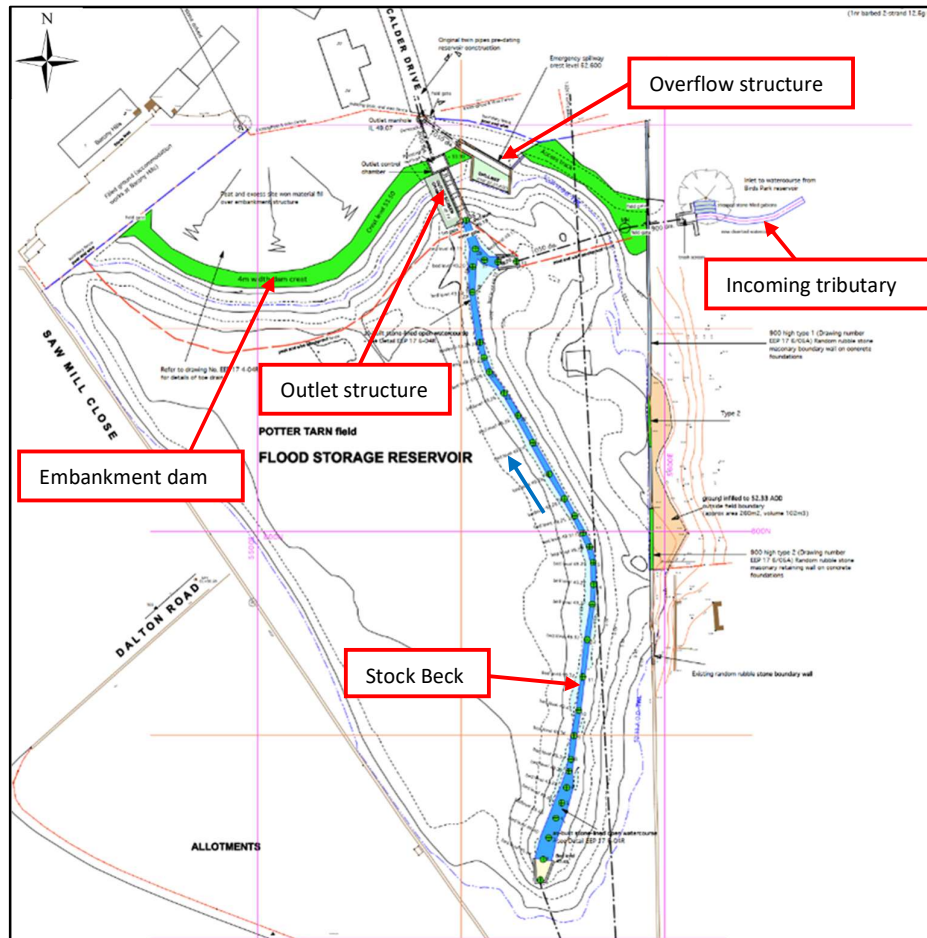


Figure 1. Stock Beck FSB – Layout Plan (NPS North West Ltd, 2008)

The embankment dam is of homogeneous earthfill construction, has a total length of 115m and a recorded height of 3.5m. The crest width is 4.0m. The faces were originally designed at slopes of 1V:3H but have been constructed at a reduced gradient. The crest and faces are grass covered.

An outlet structure is located towards the right-hand end of the dam. This conveys flows from the watercourse into the downstream surface water sewer system. An overflow structure is located adjacent to the right abutment of the dam.

Available Information

There was a reasonably good amount of information available. The Final Certificate and Certificate of Execution of Works and their associated

annexes were provided for review. However, not all the information detailed in the annexes appeared to have been transferred during the change of Undertakers. Much of this related to geotechnical investigations and construction information.

Hydrology and hydraulic modelling reports were produced during the optioneering stage and after the reservoir was constructed. There was no reporting that specifically detailed the hydraulic design of the reservoir, although there were summaries in other documents. There was a lack of detail regarding the expected performance of the reservoir, especially in terms of overtopping events. Inflows to the reservoir are from Stock Beck that flowed through a heavily urbanised catchment and a tributary to the east that had a predominantly rural, upland catchment. Flows from these had been calculated separately and it was unclear as to how they had been combined to provide the inflow for the reservoir's hydraulic design. Earlier documents indicated the reservoir was designed to cater for the 1 in 100-yr flood, but later reports reduced this to the 1 in 40-yr flood.

The construction records were limited. Although 'as-built' drawings were provided, these tended to be of a more general nature and lacked detail. There was also a lack of information regarding the sourcing and testing of fills materials. It was indicated that fill for the embankment had been excavated from the reservoir basin but, to make up a short-fall of suitable material, imported fill was also required. Much of the fill was relatively permeable in nature and a clay blanket had been provided to the upstream to provide an impounding later. However, this was indicated to extend to just above the level of the lower spillway weir.

A Prescribed Form of Record had been produced by the current Undertaker. This contained water level monitoring records for 2024 and 2025 only. In addition, detailed water level monitoring records from 2015 onwards, from electronic instrumentation installed within the reservoir basin, were provided. There was no monitoring of water levels immediately downstream of the reservoir.

The Undertaker had performed annual crest level monitoring during 2024 and 2025 and the records were provided for review. As there was no 'as-built' survey record and there were only two defined survey points on the embankment, the results were of limited value.

A representative of the Undertaker had been performing the role of Supervising Engineer 'in the spirit of the Act'. Whilst it was subsequently understood that pseudo-Section 12 statements had been produced, these were not made available at the time of the inspection.

Inspections of Small Raised Flood Storage Reservoirs

Inspection

The embankment was well-maintained and in good condition. There was no evidence of significant movement, distress, deterioration, or seepage, and there was no evidence of moles or other animal activity. However, it did not appear as tall as indicated in the reservoir records. Later interrogation of the 'as-built' records and readily available LiDAR survey information indicated that the area around the embankment toe had been infilled. The embankment crest had been constructed higher than design but much of the expected settlement had not occurred. The embankment effectively had a maximum height of 2.8m.

The outlet structure was of reinforced concrete construction with the exposed faces being masonry clad. A multi-stage debris screen was installed in the approach channel. There were three manually-controlled penstocks, one each at the inlet and outlet of the outlet chamber and a third in the outlet manhole that connected with the surface water sewer system. Only the central penstock was used to control flows, being kept in a part-open position, with the others normally fully open. They were in good condition and operated satisfactorily when tested. However, there was difficulty in ensuring the central penstock gate was returned to its correct position due to it being located within the outlet chamber and having a lack of position indicators that could be easily seen from ground level.

The spillway structure was also of reinforced concrete construction with masonry clad exposed faces. It was of an unusual design comprising lower and upper spillway weirs (Figure 2). The lower spillway discharges, via a culvert, into the outlet manhole connecting with the surface water sewer system. During more significant flood events, when the lower spillway is surcharged, excess water will discharge at the upper weir, flowing across the ground surface and along the public road system through the adjacent residential estate. Whilst the 'as-built' drawings gave the impression of a relatively flat area immediately adjacent to the upper spillway, the inspection confirmed that the ground was rising with the valley side, thus, forming a constraint to flows. Safety railing above the weir may also create a blockage risk, thus, also affecting its performance

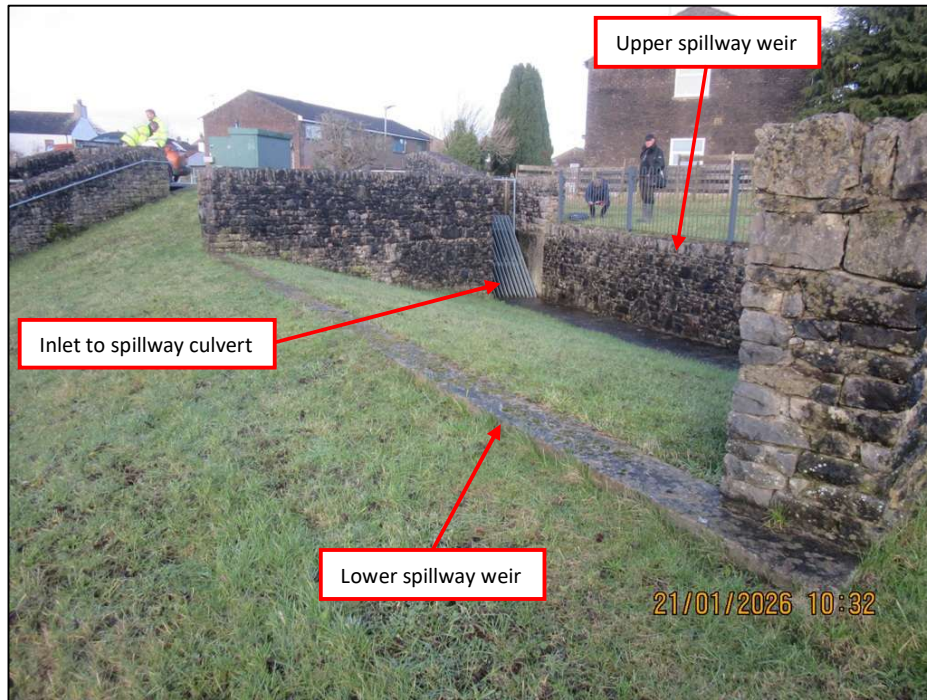


Figure 2. Stock Beck FSB, spillway structure (MM, 2026)

The instrumentation and telemetry kiosk appeared in good, operational condition. However, it was not possible to closely examine or test the equipment during the inspection.

The reservoir basin and the incoming watercourses were in satisfactory condition. The small groups of trees within the basin did not raise significant concern due to their apparent good health and the low velocities that could be expected during storage.

Water was flowing freely into the reservoir. It was noted there were no inlet controls. The Undertaker indicated that a debris screen on the incoming tributary had been known to block during storm events affecting inflows to the reservoir. Remedial works had been performed to address this risk.

Key Findings

Reservoir records

Whilst there was a reasonably good amount of information for the reservoir, some key details were lacking. In addition, there were inconsistencies between the information provided and the completed structure.

Inspections of Small Raised Flood Storage Reservoirs

Hydraulic design and performance

Records of numerous operations of the reservoir included a 'full storage' event around the time of construction completion. The reservoir withstood these successfully. However, the inspection raised several concerns:

- A lack of clarity as to the hydraulic design and expected performance.
- Discharge constraints due to capacity limitations in the downstream surface water sewer system.
- The connection of the lower spillway to the downstream surface water sewer system, thus, constraining its discharge capacity.
- Topographical constraints due to the ground profile adjacent to the upper spillway weir.

As a result, there was uncertainty as to the reservoir's performance in significant flood events and the impacts on flood rise and reservoir safety.

Structural condition and performance

The embankment dam and control structures all appeared in good condition. They had successfully withstood the impacts of previous flood storage operations and this provided confidence regarding the structural performance. However, the limited height of the upstream clay blanket raised some concern. During a significant flood event, when the upper spillway weir could be expected to operate, more infiltration into the embankment fill could be expected. An assessment of the flow through the embankment was performed. Due to the expected limited duration and scale of flows, associated with the peaks of flood operation, the risk of internal erosion was considered low.

Operational issues

Whilst the reservoir appeared in good, operational condition, the key concern related to the hydraulic performance of the reservoir as detailed above. The discharge capacity limitations of the downstream surface water sewer system provide limited opportunity to alter how the reservoir will perform during a flood event.

Due to the location of the reservoir, in relation to those responsible for it, and its small size, it can be expected to fill and substantially empty again on the passing of a flood event before the relevant people are able to attend. Therefore, responses are expected to be, primarily, of a reactive nature to deal with the after-effects of a flood operation.

Recommendations

Categorisation

The Construction Engineer for the reservoir considered that the reservoir fell within Dam Category A (ICE, 2015).

The form of the reservoir, its expected performance and its location within a residential area created debate about its categorisation. If the reservoir remained within its 'just-full' condition, it was expected to perform satisfactorily. If it was to breach in this scenario, waters could be expected to fill a low area by the embankment toe before rising and affecting the surrounding properties located at a high level. However, due to the discharge constraints in the downstream sewer system, the upper spillway is expected to operate over the full range of flood events usually considered for reservoir categorisation. This would result in overland flows affecting the downstream area which is likely to be evacuated as part of the existing flood warning procedures. An assessment of the breach flows in this scenario indicated it could still introduce a significant increase in flows and, thus, flood impact. If the downstream area was evacuated, it could be argued that there will be limited impact to life although there would be the risk of notable economic damage. However, the situation may be different if it happened at night or during extended holiday periods.

Taking all relevant issues into account, it was considered that Stock Beck FSB should be designated 'Dam Category A' (ICE, 2015).

Reservoir safety recommendations

As a result of the doubts and concerns relating to the hydraulic performance of the reservoir, three recommendations were made as to measures to be taken in the interests of safety 'in the spirit of' the Act:

- Perform a detailed topographical survey of the reservoir
- Review the hydrology for the reservoir and perform a Flood Study
- Review the topographical survey and Flood Study results, identify any remedial works required and implement these as necessary.

Other recommendations were made in relation to:

- CCTV surveys of pipework to confirm their current condition.
- Improvements to monitoring facilities.
- Improvements to aid effective operation of the reservoir.

Inspections of Small Raised Flood Storage Reservoirs

TIMPERLEY BROOK FSB

Timperley Brook Flood Storage Basin (FSB) was constructed in 1985. It is situated near Altrincham, approximately 11.5km south-west of Manchester city centre. Details of the contractor or the supervision of construction were not available.

The reservoir is an online impounding type and provides a flood storage function by controlling flows along the watercourse and storing excess water within two compartments located to each side (Figure 3). Surface areas of 66,392m² (left compartment) and 2,515m² (right compartment) were originally indicated but the Top Water Level and storage volumes associated with these were not recorded. The left compartment is normally used as a recreational park whilst the right compartment contains a wet woodland.

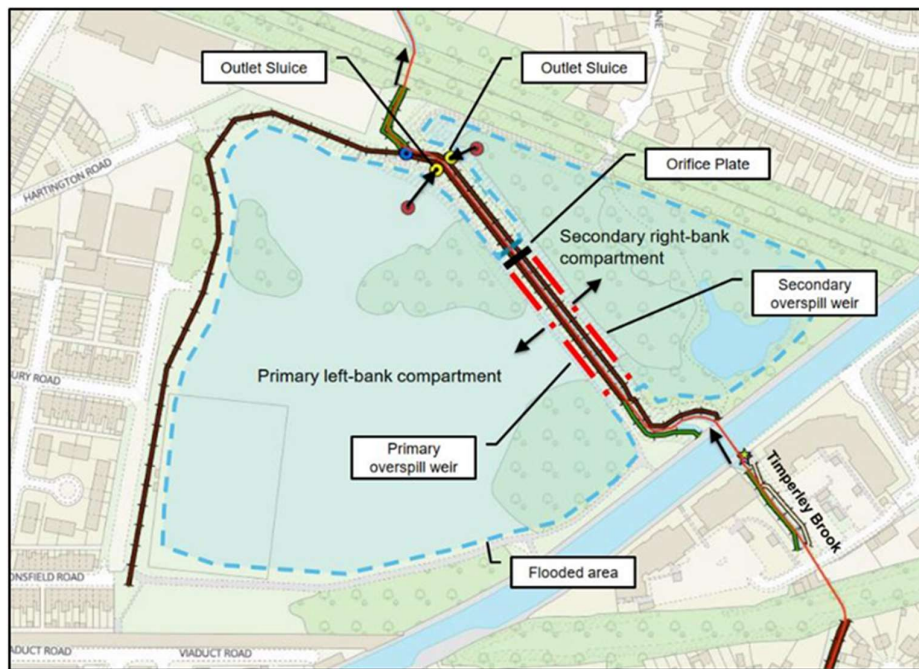


Figure 3. Timperley Brook FSB – Layout Plan (EA, 2026)

The impounding embankments are understood to be of homogeneous earthfill construction with a recorded height of 3.15m above the watercourse. The crest and faces are generally grass covered.

A sheet-piled in-channel control structure, incorporating an orifice plate and a variable level overflow weir, managed flows along the watercourse. Upstream of this, the embankments formed inlet weirs to each of the storage compartments. Penstock-controlled outlet pipes returned waters from each of the storage compartments to the downstream watercourse.

Available Information

There was very limited information available for Timperley Brook FSB. There were no design or construction records. Due to the age of the reservoir, it is likely many of the records were in paper form and, if they had existed, were no longer available. A review of historical mapping and Google Earth imagery indicated the history of the site and the reservoir's construction.

There were no records detailing the original hydraulic design and expected performance of the reservoir. A flood study for the wider catchment was carried out in 2007. Whilst this was not specifically focussed on the reservoir, it indicated the storage compartments could be expected to fill from around the 1 in 5-yr flood event. The discharge capacity of the in-channel control structure was assessed as part of the inspection and, based on the flows stated in the flood study, the storage compartments could be expected to fill between a 1 in 6-yr and 1 in 15-yr flood event. However, the Undertaker indicated the reservoir had never knowingly operated since it had been constructed. The flood study indicated the culverts on the upstream watercourse were susceptible to blockage.

A Prescribed Form of Record (PFoR) had not been prepared. There were no crest level monitoring survey points along the embankments. Electronic instrumentation had not been installed. Whilst there was a gauge board, there was no routine water level monitoring.

Whilst the staff from the Undertaker's organisation performed routine visits to the reservoir, it was not supervised 'in the spirit of the Act' and no statements were available.

Due to the limitations in the records, the Undertaker arranged for a topographical survey to be performed in December 2025 for the whole of the reservoir site. The survey results were issued at the start of March 2026. From a preliminary review, it was identified that the reservoir storage capacity exceeded 25,000m³. As a result, it was agreed that Timperley Brook FSB was a 'large raised reservoir' and that an inspection report under Section 8 of the Act was required. The Undertaker formally registered the reservoir and it was renamed 'Timperley Flood Storage Reservoir' (FSR).

Inspection

The embankments were generally well-maintained and in good condition. However, to the right compartment, there was notable tree growth and dense vegetation along the internal face and onto the crest due to the presence of the wet woodland within the basin. There was no evidence of significant movement, distress, deterioration, or seepage, and there was no evidence of moles or other animal activity. However, the embankments did

Inspections of Small Raised Flood Storage Reservoirs

not appear as tall as indicated in the reservoir records. Interrogation of the topographical survey results indicated the maximum embankment heights, at the outlet structures for each compartment, were 2.0m.

The inlet weirs to each compartment, formed within the embankment crests, were identifiable but not clearly defined. They were clear of obstructions. There did not appear to be designated spillways over the embankments for returning excess flows to the downstream watercourse.

The in-channel control structure is of sheet-pile and reinforced concrete construction (Figure 4). The primary flow control is a 1350mm diameter orifice plate. Above this, a variable height weir controls the water level rise within the upstream watercourse and, as a result, inflows to the storage compartments. The Undertaker stated that the equipment for operating the weir was no longer available and, thus, its position was now effectively fixed.



Figure 4. Timperley Brook FSB – In-Channel Control Structure (MM, 2026)

The outlets to each compartment comprise 600mm diameter pipes with upstream debris screens and downstream penstock controls. They appeared in satisfactory condition although they could not be examined internally during the visit. The penstock gates were set in a half-open position to permit surface water discharges into the watercourse. Whilst their positions could be altered to influence operation, the Undertaker stated this is unlikely to happen, as resources will be focussed on more critical assets during a flood event.

The only installed monitoring instrumentation was a gauge board fixed to the upstream face of the in-channel control structure (Figure 4). This only extended up the lower part of the face and will be submerged during a significant flood event.

The left compartment basin was in satisfactory condition. It contained several small wooded areas, but these did not raise significant concern due to their good health and the low water depths and velocities expected during a storage event. The right compartment basin was heavily wooded and had become a designated nature reserve. Although there were notable quantities of woody debris, this was not a significant concern due to the low water velocities and the trees expected to act as debris traps during storage.

Findings and Key Recommendations

Reservoir records

The lack of information regarding the reservoir had an impact on the inspection. An understanding of the reservoir's construction was gained from detailed review of historical mapping, a recent topographical survey, the knowledge of the Undertaker's representative and visual observations.

During the inspection, the confirmation that it was a 'large raised reservoir' highlighted that formal records were likely to be required, dependent on the final risk designation following registration.

Hydraulic design and performance

The flood study in 2007 indicated storage potentially commencing from around the 1 in 5-yr event. It also assessed various settings of the variable level weir at the in-channel control structure and the influence on reservoir operation. The current weir setting was close to the optimum for managing floods up to a 1 in 100-yr event. However, it was difficult to comment on the reservoir's performance as it had never knowingly operated and, given its age, this appeared to contradict the flood study results.

Flow constraints in the upstream watercourse were suspected to be influencing the reservoir's operation. In addition, the flood study in 2007 had not assessed the impacts of the flood events more usually associated with reservoirs (ICE, 2015). It was considered a new flood study, focussing on reservoir operation, was required.

Structural design and performance

The embankment dam and control structures all appeared in satisfactory condition. However, they had never been tested by a flood storage event. In addition, there was no available information regarding their construction,

Inspections of Small Raised Flood Storage Reservoirs

although there was evidence they may be constructed of locally won material. The situation raised questions about the expected performance.

Based on the anticipated construction materials, the limited duration of operation associated with a flood event and the low hydraulic gradients across the embankments, the risk of internal erosion was considered low.

Operational issues

Whilst the reservoir appeared in good, operational condition, the key concern related to the hydraulic performance of the reservoir. It was unclear when storage would commence, due to suspected upstream flow constraints, or how the reservoir would perform during exceedance events.

The small number of variable controls at the reservoir provided limited opportunity to alter how the reservoir will perform during a flood event.

During a significant flood event, the Undertaker's resources tend to be focussed on more critical assets. Therefore, responses to Timperley FSR are likely to be reactive to deal with the after-effects of a flood operation.

Recommendations

Categorisation

Although the reservoir is located within an urban area, the low height of the storage compartment embankments limits the depth of a breach and, therefore, the hazard (depth and velocity) which can credibly occur. As the reservoir is normally empty, a breach could only occur during a significant flood event. In a breach scenario, flood depths are expected to be very shallow. As a result, it was considered unlikely that lives will be at risk but damage to businesses in the immediate area may occur.

Taking all relevant issues into account, it was considered that Timperley FSR should be designated 'Dam Category C' (ICE, 2015).

Reservoir safety recommendations

As it was considered there was no significant risk to life should a dam breach occur, there were no recommendations regarding measures to be taken in the interests of safety under the Act. Other recommendations were made in relation to:

- Review the hydrology for the reservoir and perform a Flood Study.
- Detailed examination of the in-channel control structure.
- CCTV surveys of pipework to confirm their current condition.
- Improvements to monitoring facilities.
- Improvements to aid effective operation of the reservoir.
- Removal of trees from the upper part of the embankments.

SUMMARY

In the case of the example reservoirs, the Undertaker is already responsible for numerous 'large raised reservoirs' and has a good knowledge of reservoir safety legislation and its application. There is resourcing available and procedures in place to inspect, operate and maintain the reservoirs to a satisfactory standard. This may not be the case with other Undertakers, especially those that are not currently responsible for 'large raised reservoirs'.

Key findings

Even though the cases detailed in this paper could be considered good examples, the inspections highlighted key themes relating to the design and construction of 'small raised reservoirs' to be considered and addressed where required (Table 1).

Table 1. Potential design and construction issues to address

Topic	Issues potentially relevant to 'small raised reservoirs'
Hydraulic design and performance	The following may be required: • Hydrological assessments. • Hydraulic modelling assessments. • Flood studies. • Works to improve the hydraulic performance. Where the above has already been performed, there will be a need to check their validity and accuracy and update as necessary.
Construction records	Investigations to determine the reservoir's form and construction, such as: • Topographical surveys • Bathymetric surveys • Dimensional checks • Ground investigations • CCTV surveys of pipework • Structural surveys Records may require developing based on the investigation findings. Review of the records to confirm whether there are issues that could have a notable influence on reservoir safety.

Inspections of Small Raised Flood Storage Reservoirs

Topic	Issues potentially relevant to 'small raised reservoirs'
Impounding structures	Remedial works may be required, such as: • Replace / upgrade impounding structures to improve performance. • Investigation and repair of defects. • Clearance of trees and dense vegetation. • Provision of good grass cover. Works to remove / control burrowing animals.
Control structures (inc. spillways)	Remedial works may be required, such as: • Replace / upgrade control structures to improve performance. • Investigation and repair of defects. • Identification of assets that may be at increased risk of failure, blockage or other operational issues. • Improvements to operational procedures. • Increased frequency of inspection and maintenance. It is likely there will be no inlet control structures. However, where these exist, the above may still be relevant.
Hydro-mechanical equipment	The following may be required: • Replace / upgrade hydro-mechanical equipment to improve performance. • Investigation and repair of defects. • Identification of assets that may be at increased risk of failure or other operational issues. • Improved measures to confirm the operational status of equipment. • Improvements to testing and operational procedures. • Increased frequency of testing, inspection and maintenance.
Instrumentation and monitoring	Instrumentation and other facilities for monitoring the reservoir may be limited. The following may be required: • Provide / replace / upgrade facilities to enable more effective monitoring. • Investigation and repair of defects. • Consider the need for remote monitoring capabilities. • Improved measures to confirm the operational status of equipment. • Improvements to monitoring procedures. • Increased frequency of monitoring. • Review of monitoring records to confirm their accuracy and watch for trends.

Topic	Issues potentially relevant to 'small raised reservoirs'
Reservoir basins	Remedial works may be required, such as: • Clearance of trees and dense vegetation.

Issues relating to reservoir management were also identified and may need to be considered and addressed where required (Table 2).

Table 2. Potential reservoir management issues to address

Topic	Issues potentially relevant to 'small raised reservoirs'
Experience of the Undertaker	Undertakers of 'small raised reservoirs' may have limited knowledge of reservoir legislation and good practice. There may be a need to provide guidance and advice to Undertakers to make them aware of their obligations. There will be a need to confirm the details of the key personnel responsible for a 'small raised reservoir', including their position, role and contact information.
Appointment of Reservoir Panel Engineers	Following an initial inspection of a 'small raised reservoir', there may be an ongoing need for routine supervision and inspection under reservoir legislation. This will be dependent on the risk designation awarded after registration. The initial inspection should consider the Dam Category for the reservoir and other issues that could influence the risk designation.
Reservoir records	The following information and documents may need to be prepared: • Information for reservoir registration. • Preparation / update of a Prescribed Form of Record. • Preparation of a reservoir flood map • Preparation of an On-Site Emergency Flood Plan • Collation and recording of reservoir documents and drawings. Where information and documentation exists, there will be a need to review them, check their accuracy and update as necessary. Even if the reservoirs do not subsequently require the full suite of records due to their risk designation, it is recommended these are prepared and kept updated as good practice.

CONCLUSIONS

The Undertaker for the 'small raised reservoirs' detailed in this paper has experienced knowledge of reservoir safety legislation and good practices. Procedures and resourcing are in place to inspect, operate and maintain the reservoirs to a satisfactory standard. This may not always be the situation

Inspections of Small Raised Flood Storage Reservoirs

for other 'small raised reservoirs' especially where the associated Undertakers may not have previously operated under legislative requirements. Such reservoirs can be expected to have significant issues that will need to be addressed to improve their safety.

It can be expected that there will be limited information regarding the design and construction of 'small raised reservoirs'. Where it is available, the information may be outdated or there may be notable inaccuracies. There may also be a greater likelihood of reservoir safety issues to address, especially as the standards of construction expected of a 'large raised reservoir' may not have been applied and there will have been no previous requirement for inspections, operation and maintenance in accordance with reservoir safety legislation and good practices. Therefore, a higher level of input for the initial inspection can be expected as compared with routine inspections of existing 'large raised reservoirs'.

Key themes and issues have been identified based on the inspections of the example cases detailed in this paper. These are likely to also be applicable to other 'small raised reservoirs'. However, due to the variety of Undertakers and reservoir types and forms that may be encountered, different and unique findings and recommendations are guaranteed.

ACKNOWLEDGEMENTS

The authors wish to thank Martin Deane and James Penman, All Reservoirs Panel Engineers registered under the Act, for their assistance and support with performing the inspections and report production.

Acknowledgements by the authors are also given to Gary Jones-Wright, Katherine Prince, Nikki Beale and others within the Environment Agency who provided their time and assisted the inspections.

REFERENCES

- Balmforth D. (2020) *Toddbrook Reservoir Independent Review Report*. Available at *Toddbrook reservoir incident (2019): independent review* – GOV.UK (www.gov.uk)
- Balmforth D. (2021) *Independent Reservoir Safety Review Report*. Available at *Reservoir review: part B (2020)* – GOV.UK (www.gov.uk)
- Environment Agency (2026) *Policy paper, Reservoir safety reform programme*. Available at GOV.UK (<https://www.gov.uk/government/publications/reservoir-safety-reform-programme/reservoir-safety-reform-programme>)

- Griffiths J. (2012) *Written Statement – Reservoir Safety in Wales*. Welsh Government. Available at <https://www.gov.wales/written-statement-reservoir-safety-wales>
- HM Government (1930) *Reservoirs (Safety Provisions) Act, 1930, Chapter 51*. Her Majesty's Stationery Office, London, UK
- HM Government (1975) *Reservoirs Act 1975, Chapter 23*. Her Majesty's Stationery Office, London, UK
- HM Government (2010) *Flood and Water Management Act 2010, Chapter 29, Schedule 4, Reservoirs*. Her Majesty's Stationery Office, London, UK
- HM Government (2013) *Statutory Instruments, 2013 No.1590 (C.64), The Flood and Water Management Act 2010 (Commencement No.2, Transitional and Savings Provisions) (England) Order 2013*. Her Majesty's Stationery Office, London, UK
- Institution of Civil Engineers (ICE) (2015) *Floods and Reservoir Safety, Fourth Edition*. ICE Publishing, London, UK.
- ICE (Institution of Civil Engineers) (2014) *A Guide to the Reservoirs Act 1975, Second Edition*. ICE Publishing, London, UK
- UK Parliament (2017) *Post-legislative scrutiny: Flood and Water Management Act 2010, Section 4, Reservoir Safety*. [www.parliament.uk](http://www.parliament.uk/publications.parliament.uk), London, UK. Available at <https://publications.parliament.uk/pa/cm201617/cmselect/cmenvfru/990/99007.htm>



The British Dam Society

Application of the Piping Toolbox 2024 to Historic Embankment Dams: Canal & River Trust Case Studies

D WHITE, Mott MacDonald
M HEWITT, Mott MacDonald
D REBOLLO, Mott MacDonald
J SHAW, Mott MacDonald
D PRISK, Canal & River Trust
S BUTLER, Canal & River Trust

SYNOPSIS

This paper presents two case studies on the application of the Piping Toolbox 2024 methodology in assessing 18th century earthfill dams owned and operated by the Canal & River Trust. The Piping Toolbox is a systematic process used to assess the risk of internal erosion. It requires the formation of a Risk Estimation Team (RET) in order to reach consensus on the internal composition of the dam, and therefore on the credible failure mechanisms to be assessed. For these the RET articulate the failure mode description for each phase of the event tree (from initiation through to breach) and determine the quantitative failure probabilities using hole erosion test data.

This paper documents how the combination of qualitatively describing the failure modes and quantitatively calculating the failure probabilities provided insight into the key vulnerabilities to internal erosion at each dam.

INTRODUCTION

The Canal & River Trust (the Trust) is the current Undertaker for 74 large raised reservoirs across the UK. These dams originally provided water to the navigable canal network throughout England and Wales, with the majority constructed during the period between 1770 to 1830, during which more than 4000 miles of new canals which were dug (Crowe, 1994). Today, many of the 74 remaining dams continue to provide supply to the 2000 miles of restored canal network across England and Wales, while others have been repurposed for leisure activities such as sailing and/or angling.

The rapid development of high dams during this period saw many new developments in the design and construction of earthfill embankments (Skempton, 1989), in particular the progression from effectively homogenous arrangements to zoned 'Pennine-type' earthfill dams with a puddle clay core. The Trust's portfolio therefore comprises embankment dams of different compositions (Brown, 2009), with highly variable historic

Application of the Piping Toolbox 2024 to Historic Embankment Dams: Canal & River Trust Case Studies

records on the cross section, construction methodology and subsequent modifications / expansions.

Given the range of ages and the differing extent to which factual information is available to understand the arrangement and composition of the dams within their portfolio, the Trust makes effective use of Quantitative Risk Assessment (QRA) to inform its decision-making and asset management approach. Embankment dams are potentially vulnerable to internal erosion, and the Trust has applied the methodology given in the Piping Toolbox 2024 (Fell et al, 2024) to screen, assess and quantify the associated probability of failure. The outputs provide a systematic framework on which to premise risk-informed investment decisions relating to further investigation or remedial works.

THE PIPING TOOLBOX 2024

Released in September 2024, the toolbox provides a methodology for estimating the annual probability of failure of embankment dams resulting from internal erosion within the embankment, from embankment to foundation, and in the foundation. The toolbox builds on the earlier Seepage Piping Toolbox 2008, introducing a number of new concepts and methods. It is therefore relatively new to UK practice. The process for estimating the annual probability of failure by internal erosion comprises the following general steps:

1. Assemble and review all information pertinent to internal erosion and piping for the dam under assessment;
2. Screen against a list of 31 initiation mechanisms (IMs) representing potential failure modes and identify those which are credible for the specific dam arrangement;
3. For those failure modes considered credible, develop a specific event tree covering each stage of the process (initiation, continuation, progression, detection / intervention and breach);
4. Estimate the conditional probabilities for each node on the event tree, based on a combination of quantitative analysis (including interpretation of Hole Erosion Test (HET) data (Wan & Fell, 2004) to determine whether erosion would initiate under a given hydraulic shear stress), historic dam failure rates and the qualitative failure mode description;
5. Combine the event tree probabilities to calculate the annual probability for each failure mode, and use this to inform follow-on risk analysis and decision-making.

The methodology is explained in further detail in Section 2 of the Piping Toolbox guidance document.

APPLICATION OF THE PIPING TOOLBOX TO THE TRUST'S DAMS

The quality and credibility of a quantitative risk estimate is contingent on understanding of the history, composition and arrangement of the dam under assessment. Undertaking the Piping Toolbox therefore requires an initial phase of assembling and interpreting the available historic records, GI data and monitoring information. Detailed construction records are no longer available for many 18th and 19th century dams, and it is therefore necessary to be aware of the typical design details and construction methodologies available at the time. In order to promote scrutiny of the interpretation, and to mitigate individual bias, a Risk Estimation Team (RET) is formed (Reclamation, 2019b) which comprises a multidisciplinary team including All Reservoir Panel Engineer(s), the Supervising Engineer (also representing the Undertaker) and geotechnical specialists. The RET's objective is to reach consensus on: (i) composition and condition of the dam and any penetrating structures, i.e. conduits; (ii) the failure mode descriptions and corresponding event trees, and (iii) estimation of the conditional probabilities for each node on the event tree.

CASE STUDY: SLAITHWAITE RESERVOIR

Slaithwaite reservoir is recorded as having been constructed between 1795 and 1799. The dam is reported to have originally been 21m high, later raised to 22m to increase freeboard (Binnie, 1987). No drawings of the original construction of the dam exist; however, the available records (from the 1930s) noted the presence of a central clay core within the embankment supported by shoulder fill on the upstream and downstream faces.

A reinforced concrete spillway is present at the left abutment (Figure 1). Major modifications to the spillway were undertaken in the early 1970s including extending the weir, lowering the invert and widening the upstream end of the channel. The remaining upper spillway channel was widened as part of remedial works in 1991/1992.

Application of the Piping Toolbox 2024 to Historic Embankment Dams: Canal & River Trust Case Studies

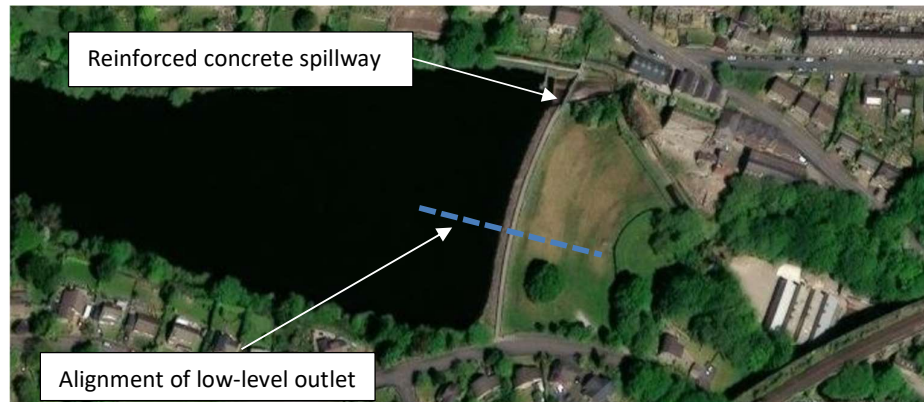


Figure 1 – Slaithwaite Layout Plan

A single low-level outlet passes through the dam at approximately its midpoint, comprising a cast iron pipe passing through the central core. Downstream of the core the pipe passes through a masonry bulkhead and continues within a masonry culvert until it reaches the downstream shoulder (Figure 2). The dam has a history of seepage emerging into the masonry outlet culvert near the headwall.

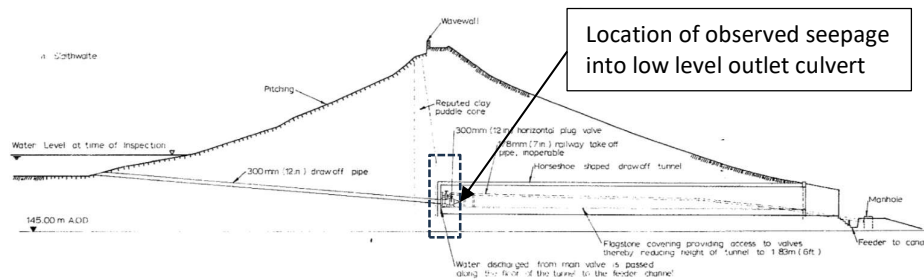


Figure 2 - Indicative Slaithwaite Embankment Cross Section (1978)

Upon review of the seepage monitoring data against reservoir level, a step-change (increase) was observed when reservoir levels rose above 2-3m from top water level (TWL). In order to gain insight into the origin of the seepage (and the associated risk of internal erosion), and to identify other potential vulnerabilities within the embankment, the Trust undertook a Piping Toolbox with support provided by Mott MacDonald.

The Piping Toolbox methodology was used to identify (screen) the credible failure mechanisms and highlight any information gaps, and hence inform the scope of a ground investigation. On the basis of the gaps identified by the Toolbox, ground investigations were specified comprising boreholes through the crest and downstream shoulder including permeability testing of the foundation as well as installations to better understand the phreatic profile. A PRIME geophysical survey was also undertaken in order to

determine variations in resistivity across the dam and hence identify areas of higher saturation and/or seepage flows (Butler et al, 2024).

For the credible failure mechanisms which had been screened in, the RET articulated each stage of the event tree, from initiation of erosion through to dam breach. In doing so the team considered the available historic records, intrusive investigation and ongoing monitoring data, as well as the likely construction methodology followed. This yielded the following insights:

- **Embankment construction:** The ground investigation found relatively little variation in material characterisation across the embankment fill, and therefore (contrary to the archive records) did not identify a clearly zoned central core. Samples taken under the dam crest indicated a clay of finer grading and lower stiffness compared to those on the downstream slope. Having considered the potential construction methodology, the RET consensus was that the clay fill had likely been selectively placed, with more carefully controlled puddling of the central 'core'. However, the similar material nature of the adjacent shoulder fill (also formed of clay) means that the embankment is likely to behave in a homogeneous manner, therefore screening out mechanisms relating to differential settlement between the core and shoulder zones;
- **Dam foundation:** While the majority of the embankment is founded on rock, a layer of highly weathered mudstone was identified under the right abutment, at a similar elevation to the low level outlet culvert (Figure 3). The RET considered that a through-going crack or defect in this material could result in a potential seepage pathway, which may be presenting itself as seepage into the low level outlet culvert; however subsequent analysis of the HET results indicated that the driving head would be insufficient for erosion to initiate and hence the risk was deemed to be very low. The ground investigation did not encounter any evidence of the embankment fill extending into the rock foundation to form a cutoff, and therefore the presence of continuous defects close to the surface of the rock formation under the embankment could not be ruled out;
- **Outlet culvert:** The ground model developed for the assessment showed the outlet culvert within the embankment fill, significantly above the rockhead (Figure 3). Given its internal condition and the lack of evidence of settlement, the RET considered that the culvert is likely to be supported either on pillars above the formation or on a localised rock ledge, increasing the potential for differential settlement occurring at the interface with the surrounding embankment and consequently forming a low stress zone through which hydraulic fracture could occur;

- **Spillway:** It was suspected that the spillway widening works of the 1970s and 1990s may have extended beyond the original rock foundation and been, in part, founded on embankment fill to the RHS. Therefore, there was a risk of the embankment fill beneath the spillway weir and channel settling away from the spillway slab resulting in the formation of a seepage pathway. The geophysics survey identified significant changes in resistivity at high water levels, specifically within the 3m range up to TWL. The resistivity changes were concentrated at both abutment locations with the most prominent changes recorded at the left abutment (adjacent to the spillway).

Table 1 – Credible Failure Modes Identified for Slaithwaite Dam

Grouping	IM Reference and Description
Upper Embankment	IM1: Cracking resulting from cross-valley differential settlement IM7: Desiccation cracking at the crest of the embankment (note: found to be not significant based on HET results, i.e. erosion would be unlikely to initiate) IM8: Desiccation cracking at the intermediate crest during embankment staging (note: found to be not significant based on HET) IM13: Transverse cracking caused by earthquake
Lower Embankment	IM12: Cracking over irregularities in foundation profile

Grouping	IM Reference and Description
Foundation	IM26: Erosion of crack in extremely weathered rock foundation (note: found to be not significant based on HET) IM27: Open or in-filled defects in a rock foundation IM29: Contact erosion into open defects in a rock foundation
Conduit	IM18: Crack or gap around the low level outlet pipe and culvert, continuous through the embankment IM19A: Erosion into the low level outlet culvert
Spillway	IM21: Crack/gap adjacent to the spillway wall resulting from the wall detailing IM22: Crack/gap adjacent to the spillway wall resulting from settlement of the adjacent embankment fill

CASE STUDY: SLIPPER HILL RESERVOIR

Slipper Hill Reservoir was recorded as being built in 1796 as a 'temporary' reservoir and subsequently altered/enlarged in 1832 to form the current reservoir (Binnie, 1987). The embankment is up to 6.5m high, formed of zoned earthfill with a central puddle clay core. A low level outlet structure is present within the dam, comprising a masonry culvert / wetwell within the upstream face, with flow conveyed via a cast iron pipe which passes through the central core and discharges into a downstream culvert.

Seepage had been observed entering into the downstream culvert via the side walls and soffit, although monitoring of seepage flows at particular emergence locations did not identify an increasing trend. The risk of internal erosion relating to this structure was raised as a concern in the 2023 Section 10 Inspection, specifically due to the expected high hydraulic gradients and multiple interfaces between the central core and adjacent (relatively stiff) masonry structures.

Application of the Piping Toolbox 2024 to Historic Embankment Dams: Canal & River Trust Case Studies



Figure 4 – Slipper Hill upstream masonry wetwell at low level outlet



Figure 5 – Slipper Hill downstream masonry portal at low level outlet

To better understand the risk of dam failure through a Piping Toolbox assessment, the Trust commissioned a ground investigation comprising 12 No. boreholes across the crest and downstream face of the dam, trial trenches across the crest and trial pits localised around the outlet structure portals. Interpretation of the ground investigation results identified the presence of a distinct puddle clay core within the centre of the embankment, between the upstream and downstream masonry structures. However, given the age of the dam, no records were available to verify the detailed arrangement of the outlet conduits; intrusive investigation of the embankment fill immediately surrounding the conduit proved challenging from a reservoir safety perspective as it risked creating a potential seepage pathway at a vulnerable location. It was therefore necessary to reach a consensus amongst the RET on the inferred arrangement, which yielded the following insights:

- **Support to cast iron pipe:** The relative levels of the wet well, downstream culvert and foundation profile as interpreted under the structure led the RET to infer the presence of a masonry support under the outlet pipe. This was further supported by CCTV survey data within the cast iron pipe which showed no observable deformation or settlement. On this basis, the size of penetration through the embankment would be large relative to the embankment height, with the potential for a low stress zone to form around the conduit. The mechanism for a low stress zone and hydraulic fracture through the core material immediately around the exterior of the conduit was therefore considered to be credible;
- **Interface between core and masonry structures:** The width of the core was investigated by means of trial trenches across the crest. Based on the offset between the upstream wetwell and downstream culvert, the RET inferred that the core directly abutted the (stiff) masonry structures

(Figure 6), with the potential for arching to occur over time as the core settled relative to these. It was noted that any support to the core material provided by an (inferred) underlying masonry footing would limit the extent of any differential settlement occurring;

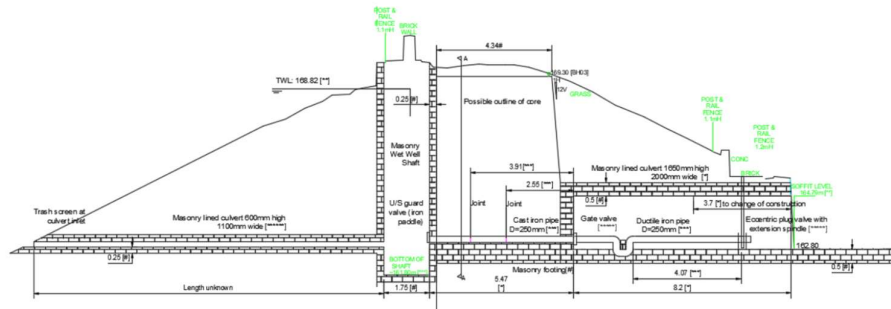


Figure 6 – Interpretative cross section of low level outlet at Slipper Hill

- Leakage out of pressurised pipe:** As discharge through the outlet was typically controlled via a downstream valve, stability failure of the downstream embankment face as a result of leakage from an open defect in the cast iron pipe was considered. Based on the CCTV internal condition survey of the pipe, the RET consensus was that this mechanism was potentially credible but had low significance (low failure probability);
- Seepage into downstream culvert:** Seepage pathways emerging into the outlet culvert were found to be not significant given the fixed defect opening size within these structures (which would prevent gross enlargement of a seepage pipe through the embankment) relative to the grading of the adjacent core material. It was further noted that these would be readily observable during periodic visual inspections within the culvert, increasing the prospects for detection and intervention.

These findings were incorporated into the conditional probability calculations for each failure mode that had been screened in:

Table 2 – Credible Failure Modes Identified for Slipper Hill Low Level Outlet

IM Grouping	IM Reference and Description
Embankment	IM10: Arching of the core onto the adjacent masonry structures

Application of the Piping Toolbox 2024 to Historic Embankment Dams: Canal & River Trust Case Studies

IM Grouping	IM Reference and Description
Conduit	IM18: Crack or gap around the cast iron pipe as it passes through the core
	IM19A: Erosion into the masonry culvert
	IM19B: Erosion due to leakage out of the cast iron pipe
Foundation	IM26: Erosion of crack in soil foundation
	IM28: Contact erosion with a coarse soil layer within the foundation

DISCUSSION

When using the 2024 Piping Toolbox, the quantitative failure probabilities for many of the assessed mechanisms were found to be high; in some cases several orders of magnitude greater than the RARS Tier 2 outputs for the same dam.

The Hole Erosion Test (HET) data obtained from the ground investigations supported improved assessment for both dams. HET test data for the specific material / composition of the dam under assessment had a significant influence on the output probabilities for many of the failure mechanisms. Where site-specific HET data is not available, a suggested range of I_{HET} and critical shear stress values are provided in the 2024 Piping Toolbox methodology based on soil classification (refer to Table 7.10 of the Piping Toolbox guidance document). However it was found that the critical shear stress obtained from testing is considerably greater than the values suggested in Table 7.1 of the Piping Toolbox, which may be overly conservative when applied to typical UK clay core dams. This reinforces the advice in the Toolbox to undertake site specific testing.

The RET view was that the quantitative outputs from the 2024 Piping Toolbox appeared too high where related to defects which would have been present since original construction of the dams (~200 years ago). The ranking of the assessed failure modes for each individual dams was considered more useful than the absolute probabilities with regards to informing subsequent decisions. The process of reaching consensus amongst the RET by articulating the qualitative detailed failure mode descriptions was found to be highly valuable in understanding potential vulnerabilities at each dam, and allowed a number of key failure mechanisms to be ruled out due to low significance and plausibility.

Prior to undertaking the Piping Toolbox assessment at **Slaithwaite**, it had been postulated (based on drainage flow response to reservoir level change)

that leakage was generally occurring in the upper part of the core; initial plans to address this considered installation of sheet piling along the full dam crest. However, screening of credible failure modes using the Toolbox highlighted specific locations for potential seepage-related vulnerabilities. The combination of: (i) the interpretation of the survey and monitoring data (in particular the geophysics resistivity findings); (ii) the qualitative process of describing the individual failure mechanism steps, and (iii) the comparative quantitative failure probabilities indicated that the observed seepage was originating close to the spillway on the left abutment. Following completion of the Piping Toolbox, subsequent investigations of the rock formation under the spillway revealed the presence of a near-vertical interface between the rock and embankment fill. This coincided with the locations where the spillway had been widened throughout the 20th century, resulting in a creation of a seepage pathway due to the embankment fill settling away from the rock face. As a result of the targeted investigation, remediation of this vulnerability was completed in 2025 via a programme of grouting works.

At **Slipper Hill**, the quantitative failure probability findings demonstrated that mechanisms relating to seepage into or leakage out of the low level outlet pipe and downstream culvert were of low significance or plausibility. However, the differential settlement between the penetrating structure and the surrounding embankment could create a vulnerability in the form of low stress zones through which hydraulic fracture could occur. This informed the subsequent decision-making by the Trust when considering appropriate risk reduction measures (including enhanced monitoring or remedial works) in terms of their potential impact at each stage of the event tree.

CONCLUSION

The quantitative outputs from the 2024 Piping Toolbox appear high (overly probable) where related to defects which would have been present since original construction of the dams some 200 years ago. Some of the failure mechanisms assessed were found to be several orders of magnitude higher than the Tier 2 RARS outputs for the same dam.

Supplementary guidance would be beneficial to better align the Piping Toolbox for application to historic UK embankment dams, in particular the estimation of critical shear stresses to be applied where site-specific Hole Erosion Test (HET) results are not available.

The process of reaching consensus amongst the Risk Estimation Team (RET) by articulating the qualitative detailed failure mode descriptions was found to be valuable in understanding potential vulnerabilities at each dam; in

Application of the Piping Toolbox 2024 to Historic Embankment Dams: Canal & River Trust Case Studies

each case these insights allowed risk reduction measures (such as enhanced monitoring or remedial works) to be more clearly targeted.

The Piping Toolbox methodology provided additional benefits to the reservoir Undertaker: (i) the outputs included a documented thorough interpretation of all available data that was then assessed in detail by the RET; (ii) this documented and peer-reviewed data set was subsequently used to support other reservoir safety assessments such as updating the Tier 2 risk assessment and informing subsequent spillway and conduit investigations, among others.

REFERENCES

- Binnie (1987). *Early dam builders in Britain*. Thomas Telford, London, UK
- Brown (2009). Canal reservoirs in Great Britain. *Proceedings of the Institution of Civil Engineers - Engineering History and Heritage* **162** (2): 103–109
- Butler et al (2024). Novel geophysical ground imaging technology for the automated long-term monitoring of reservoir dams. In *Managing Risks for Dams and Reservoirs. Proceedings of the British Dam Society Conference*. British Dam Society, London , UK
- Crowe (1994). *English Heritage Book of Canals*. B.T. Batsford / English Heritage
- EA (2013). *Guide to Risk Assessment for Reservoir Safety Management (RARS)*. Environment Agency, Bristol, UK
- Fell, Foster, Cyganiewicz, Davidson and Sills (2024). *Methods for Estimating the Probability of Failure of Embankment Dams by Internal Erosion and Piping*. The University of New South Wales, Sydney, Australia
- Reclamation (2019b) *Risk Management – Best Practices and Risk Methodology*. United States Bureau of Reclamation, Washington, DC, USA.
- Skempton (1990). Historical development of British embankment dams to 1960. In *Clay Barriers for Embankment Dams*, Thomas Telford, London.
- Wan & Fell (2004). Investigation of rate of erosion of soils in embankment dams, *Journal of Geotechnical and Geoenvironmental Engineering*, **30(4)**: 373-380



The British Dam Society

Hydraulic Harmony: Balancing Safety and Supply at Lough Island Reavy

DAVID BELL, AECOM

SARAH WATSON, AECOM

SYNOPSIS

The Lough Island Reavy Reservoir Refurbishment project represents a critical engineering achievement aimed at enhancing the safety and resilience of the reservoir while maintaining uninterrupted service.

Key components of the refurbishment included the reconstruction of the spillway channel to enhance its hydraulic efficiency and structural integrity. This involved the use of advanced hydraulic modelling and engineering techniques to manage flow rates and mitigate flood risk during construction. Additionally, two 500mm diameter siphons were installed to significantly increase the reservoir's drawdown capacity, ensuring compliance with modern safety standards and providing a reliable mechanism for water level management during maintenance or emergencies.

The project also prioritised supply security by incorporating innovative design solutions that allowed the reservoir to remain operational throughout the works. This was achieved through meticulous planning, phased construction, and the deployment of temporary bypass systems to maintain raw water delivery to downstream users.

This technical paper explores the challenges, methodologies, and outcomes of the Lough Island Reavy Reservoir Refurbishment, offering valuable insights into the complexities of upgrading critical water infrastructure without disrupting essential services.

RESERVOIR DETAILS

Lough Island Reavy Reservoir, situated near Kilcoo in Northern Ireland as shown in Figure 1, was constructed in 1840 to provide a reliable water supply as part of the wider Mourne Catchment system. The Northern Ireland Water (NI Water) owned reservoir, which was formed by the raising of the original lough, has a capacity of 9091 ML, surface area of 105 ha and a length of 2 km.

Hydraulic Harmony: Balancing Safety and Supply at Lough Island Reavy

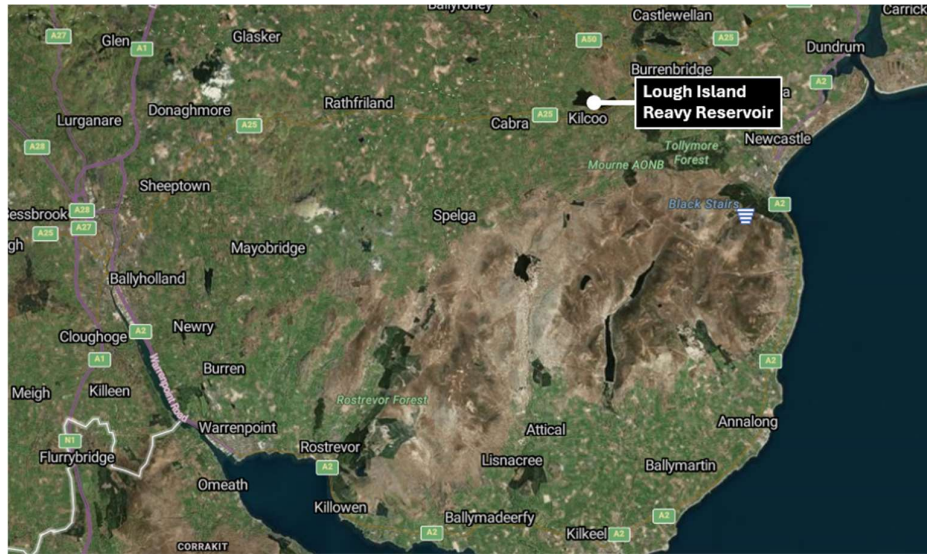


Figure 1. Reservoir Location (Map Source: Bing Maps)

The reservoir comprises four earth embankments (South, East, West and Northwest) with a maximum height of 12.5 m, and a precast concrete wave wall along the East embankment crest. Downstream slopes were originally 2:1 to 6.1m below crest, flattening to 2.5:1, with rockfill berms later added to the South and East embankments. Upstream slopes are 2.5:1 to 6.1m below crest, flattening to 3.0:1. The overflow, located close to the toe of the South embankment, consists of a 19m straight concrete weir discharging to a 4.5m wide masonry channel. The valve tower houses three 450mm draw-off valves and one 450mm scour valve; the draw-off valves feed a 600mm outlet supply pipe. Outlet pipework runs in a culvert from the tower base to the outlet chamber at the downstream toe of the South embankment. The reservoir arrangements are summarised in Figure 2.



Figure 2. Reservoir Arrangement (Map Source: Bing Maps)

RESERVOIR LEGISLATION

Although there remains no formal reservoir legislation in Northern Ireland, NI Water have been acting in the spirit of the GB Reservoirs Act 1975 by undertaking Section 10 Inspections at the reservoir since the introduction of this legislation. The 2019 Section 10 Report, which forms the basis of the works set out in this paper, categorised each dam in Category A as defined in the Floods and Reservoir Safety 4th Edition (ICE 2015) (F&RS), where a breach could endanger lives in a community.

SPILLWAY DETERIORATION

The masonry spillway channel has deteriorated significantly over time, with substantial voids beneath the floor and behind the side walls. This has caused discharge flows to exit the channel through the base, passing underneath and re-entering downstream. In the 2019 Section 10 Report, Inspecting Engineer Robert Mann made the following recommendation:

The voiding appears to have progressed to a degree that the spillway has lost much of its integrity for retaining water. A recommendation is made under 12.2 Measures to be taken in the interests of safety under section 10 (3) (c) to restore or replace the spillway channel to achieve adequate structural and water-retaining integrity and safely discharge the appropriate flood flows. Include for tracing and sealing the adjacent leakage from the area of the eroded part of the approach channel from the Muddock Feeder.



Figure 3. Spillway Deterioration

Hydraulic Harmony: Balancing Safety and Supply at Lough Island Reavy

EMERGENCY DRAWDOWN AND SUPPLY DEFICIENCIES

As a Category A reservoir, the basic recommended drawdown requirement is 5% height per day (% h/d), as set out in the guidance provided by the Environment Agency entitled 'Guide to drawdown capacity for reservoir safety and emergency planning' (2017). The drawdown arrangements include a 450mm diameter scour outlet, supplemented by a 600mm diameter tee off the supply pipe, with a combined Installed Drawdown Rate of 2 % height per day (h/d). The following recommendation was made within the 2019 Section 10 Report:

I recommend under 12.2 Measures to be taken in the interests of safety under section 10 (3) (c) to study options for increasing drawdown rate at the reservoir to meet current UK guidelines, and implement a preferred option unless judged to be disproportionate having regard to a risk-based study.

Further to this, due to the strategic importance of the raw water source, two pumping stations are positioned downstream of the South embankment, each serving distinct water supply regions from the main 600mm diameter outlet pipe. Historically, there have been challenges with the simultaneous operation of both pumping stations, with the first station on the line 'starving' the second when both are in use. A further requirement of the scope to increase the drawdown rate at the reservoir was to establish an isolated supply for each pumping station.

DESIGN BASIS AND DEVELOPMENT

AECOM was appointed by NI Water to design, manage, and oversee a project to replace the existing spillway in accordance with modern design standards, while also enhancing drawdown capacity and providing an isolated supply for each pumping station.

Spillway Reconstruction

The design development of the proposed spillway replacement was guided by the assessment of several critical factors, as detailed further in the following text:

Hydraulic Capacity: The reservoir has a limited direct catchment of 170 ha (of which 105 ha is the reservoir surface area), with the majority of inflow via two indirect arrangements totalling 1116 ha. Evaluation of the reservoirs hydraulic performance was undertaken in the form of a formal Flood Study and Computational Fluid Dynamics (CFD) model. The Flood Study highlighted the significant attention provided by the reservoir, due to the surface area of the reservoir in relation to the overall catchment area, with the flood hydrographs shown in Figure 4.

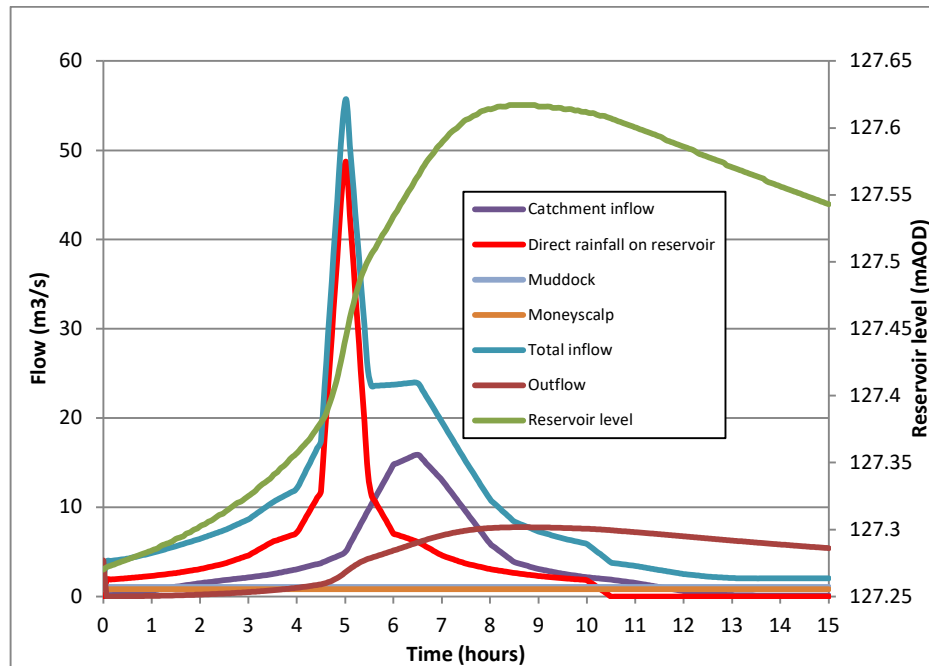


Figure 4. Safety Check Flood (PMF)

The CFD analysis of the spillway was undertaken for a range of operational and flood scenarios, including sensitivity tests on surface roughness to represent both existing and proposed conditions. The results identified overtopping of the existing channel at the downstream end of the chute, which informed the design of the proposed arrangement. Peak velocities exceeded 10m/s (see Figure 5).

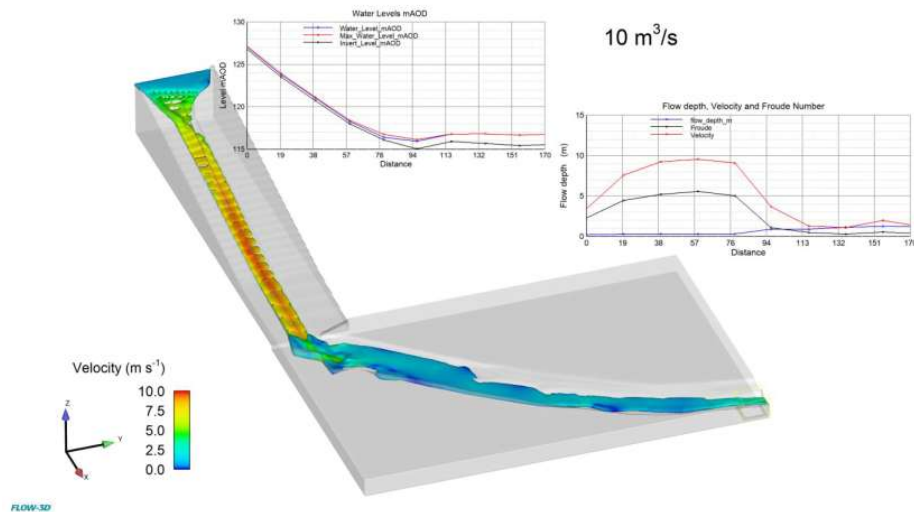


Figure 5. Computational Fluid Dynamics Analysis

Hydraulic Harmony: Balancing Safety and Supply at Lough Island Reavy

Structural Integrity and Durability: Analysis of the existing spillway's structural integrity, foundation performance, and overall residual life were undertaken to determine the feasibility of refurbishment and replacement. Cores taken during the Early Contractor Involvement (ECI) stage confirmed significant voiding beneath the channel, placing the structure beyond viable in-situ repair. Masonry, masonry-lined, and reinforced concrete options were explored, with reinforced concrete the preferred solution for its robustness, predictable flood performance, construction quality control, and ease of maintenance. The design incorporated a no-fines concrete underlayer with associated underdrainage to relieve uplift pressure, plus strategically located shear keys to control under-seepage and sliding in line with site geotechnical conditions.

Operational Reservoir Requirements: No intentional lowering of water level was permitted during the construction works. To accommodate this constraint, a strategic construction sequence was implemented to utilise the permanent siphon installation as a temporary bypass, as described later.

Proximity to Impounding Embankment: The design development evaluated the spillway's proximity to the South embankment, including potential effects on stability and construction-related vibration. Further to this, the spillway alignment was confined to the footprint of the existing channel.

Site Constraints: A thorough evaluation of site-specific constraints was undertaken in advance of the construction works, including access and weight limits, existing utilities, environmental controls, temporary works, sequencing, and seasonal windows, while maintaining public access for angling and amenity. The spillway sits within 5m of the main South embankment toe, with the proposed siphon pipes occupying the corridor between the spillway and the embankment, requiring careful planning to safeguard the embankment and appurtenant structures.

Material Re-Use: Opportunities for material re-use were prioritised to reduce embodied carbon and waste. The design repurposed masonry from the existing spillway channel for shoreline protection elsewhere around the reservoir.

Siphon

The proposed siphon was to serve dual functions: increase the reservoir's drawdown capacity to comply with current safety standards and secure supply to the two downstream pumping stations.

The following scheme requirements were set out for the siphon design:

- Operating range: Upper 4.5m
- Supply Capacity: 425 l/s across the full operating range
- Drawdown Capacity: 500 l/s at 1m below TWL

The siphon design was undertaken in line with *CIRIA C813 Siphons in dams – design, installation, operation, management and testing*, with some of the key design considerations described further in the following sub-sections.

Hydraulic Capacity: After assessing multiple diameters and configurations, a dual 500mm arrangement was selected with a cross-connection for resilience. One siphon was dedicated to emergency drawdown, with the second siphon primarily for supply while providing additional emergency drawdown capacity when required. The design evaluated conveyance across the full head range, priming performance, air management, head losses, and surge behaviour during start-up and shut-down.

Structural Integrity and Durability: A robust pipe material selection assessment was undertaken, considering siphon pressures; thrust and restraint requirements; pipe support and anchoring; fatigue under cyclic operation; corrosion protection and linings. High-density polyethylene (HDPE) pipe was selected with ductile iron (DI) fittings in the priming and discharge chambers. To navigate an existing bridge, a short above-ground section was required, with the operating HDPE pipe encased within an oversized DI carrier for protection. Within the reservoir, the siphon's upstream leg incorporated anchor blocks and mattresses to resist buoyancy uplift.

Proximity to Impounding Embankment: Three siphon alignments were evaluated during the design development as shown in Figure 6: (1) a route through the dam body to minimise overall pipe length; (2) a crest alignment over the top of the dam following the draw-off pipework; and (3) an alignment around the abutment, routed along the corridor between the spillway and the embankment.

Option 3 was selected, despite being the longest length, to maintain physical separation from the main body of the dam, thereby avoiding new penetrations and minimising the scope and cost of associated support and protection measures. Through the abutment, the pipelines were encased in concrete to control seepage and, as a secondary benefit, provide additional thrust restraint adjacent to the priming chamber.

Hydraulic Harmony: Balancing Safety and Supply at Lough Island Reavy



Figure 6. Siphon Route Optioneering

Operational and Resilience Requirements: Consistent with the spillway constraints, no intentional reservoir level reduction was permitted during construction.

To minimise routine operational demands, the siphons were designed as self-priming, with the pipe crown set below normal top water level, supplemented by a secondary pumped priming chamber located just off the dam crest. The supply siphon was intentionally oversized to accommodate potential future pump capacity increases at the downstream pumping station.

An inlet screen mitigates debris ingress.

Site Constraints and Constructability: As per the overflow construction, constructability planning was critical. Early engagement with the Contractor enabled the design team to identify, assess, and mitigate risks associated with the limited working area. To maintain the reservoir level, all upstream works were planned to be undertaken by engineering divers.

Combined Drawdown Capacity

A review of the combined capacity of the outlets, in line with the Environment Agency guide entitled '*Guide to drawdown capacity for reservoir safety and emergency planning*' (2017), concluded that the existing arrangements ($2.98\text{m}^3/\text{s}$) were grossly inadequate, equating to 2% h/d in comparison to the recommended drawdown requirement of 5% h/d, with a shortfall of circa $4.5\text{m}^3/\text{s}$ at top water level.

Having exhausted options to increase scouring through the existing pipework and determining that a substantially larger siphon or multiple siphons to achieve the full drawdown rate was not economically disproportionate, an integrated discharge strategy was developed to meet the emergency requirement.

To meet the required rate over the upper operating range of the reservoir, the spillway arrangements were enhanced to incorporate four penstocks in the spillway as shown in Figure 7, with the solution developed through a structured value engineering process.



Figure 7. Spillweir Penstocks

CONSTRUCTION

BSG Civil Engineering was appointed by NI Water to deliver the construction of the project, over a 16 month period.

Pre-planning and Scheme Development: A crucial factor in the success of the construction phase was the thorough pre-planning and scheme development conducted during the ECI stage, aimed at preserving reservoir operations and minimising impact on water levels. The two elements of the scheme, the spillway and twin siphon pipes, while conflicting, were planned to be constructed in harmony. If planned independently, a considerable extension to the construction programme would have been necessary, along with additional temporary measures.

Flow Diversion: A key decision was to use portions of the permanent siphon pipework as a temporary bypass during the spillway construction. The construction sequence commenced with the installation of the downstream leg of the twin siphon, which served as a dependable temporary bypass for flows that would typically go through the spillway, as illustrated in Figure 8. This bypass was tested to confirm its capacity and operational control prior

Hydraulic Harmony: Balancing Safety and Supply at Lough Island Reavy

to the commencement of spillway works. Once the flows were redirected, a safe, dry, and stable work zone was established for the spillway construction.

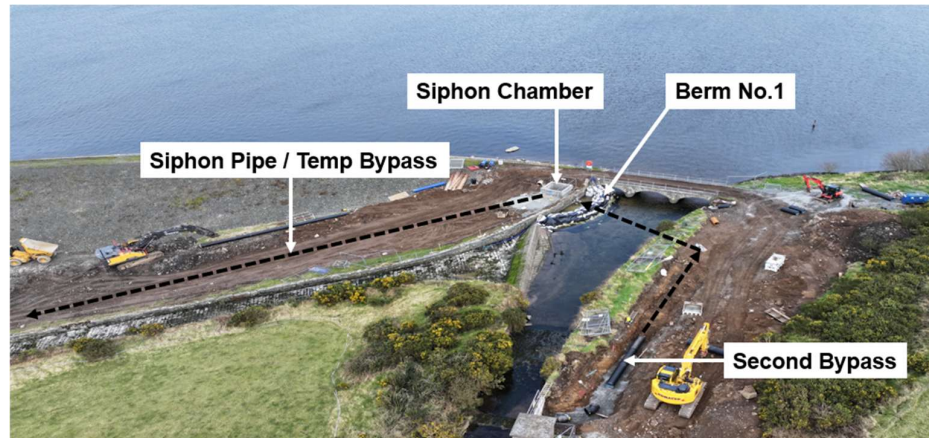


Figure 8. Temporary bypass using drawdown siphon pipes

Spillway Construction: Another critical safety consideration during the construction phase was the sequencing of the spillway construction to limit the length of exposed foundation to a maximum of 12 m, corresponding to two spillway invert bays. The construction sequence included the following steps; (1) excavating the existing channel to the foundation level; (2) preparing the foundation; (3) installing a no-fines concrete underlayer and drainage pipework to enhance drainage and mitigate uplift pressures; and finally (4) constructing the structural components of the spillway, in a phased manner, with construction squads working in tandem as illustrated in Figure 9.



Figure 9. Phased Spillway Construction

Siphon Installation: The installation of the upstream leg of the siphon and its downstream connections were undertaken on completion of the spillway channel works. Engineering divers conducted the underwater installation of intake structures and associated screens. To expedite the installation process, the pipework and anchor blocks were connected on dry land, as shown in Figure 10, and then lifted into the reservoir using airbag support.



Figure 10. Siphon Pipe Preparation

Once in position, the airbags were deflated, allowing the pipework to be lowered to the bed in a controlled manner as shown in Figure 11. Underwater inspections and pressure testing were performed to verify the functionality of the twin siphon pipework.



Figure 11. Siphon Pipe Installation

Commissioning

Upon completion of the works, a comprehensive commissioning exercise was conducted to ensure that all elements operated effectively and met the design specifications. This commissioning process included; (1) operation of the scour and supply siphons at elevated and reduced water levels; (2) operation of the supply siphon with a varied downstream pumping rate; (3) operation of the spillweir penstocks; and (4) evaluation of the flow patterns, aeration, and energy dissipation characteristics of the spillway

Hydraulic Harmony: Balancing Safety and Supply at Lough Island Reavy

channel under a range of flow conditions. All elements performed as expected, with the completed scheme shown in Figure 12.



Figure 12. Completed Scheme

ACKNOWLEDGEMENTS

NI Water, the reservoir owner, with special thanks to the capital delivery, operational, reservoir safety, and asset management teams.

BSG Civil Engineering, the contractor. Their expertise in managing the construction processes, adhering to timelines, addressing challenges as they arose, and maintaining safety standards significantly contributed to the project's success.

The wider project team, including the Qualified Civil Engineer, engineers, consultants, and specialists, whose collective efforts and diverse expertise enriched the project.

The local community for their positive engagement. Project meetings were conducted at the local sports team's clubhouse, which provided mutual benefits to both the club and the project.

The combined efforts of all have resulted in a successful outcome that meets the needs of the community and enhances the operational capabilities of the reservoir.

REFERENCES

CIRIA C813 (2023). *Siphons in dams – design, installation, operation, management and testing*. CIRIA, London, UK

Environment Agency (2017). *Guide to drawdown capacity for reservoir safety and emergency planning*. Environment Agency, Bristol, UK



The British Dam Society

Protecting Wales' largest reservoir: dam safety and lasting resilience at Llyn Celyn

S GOLDS, Mott MacDonald Bentley (MMB)

G CORNELIUS, Mott MacDonald Bentley (MMB)

M EDMONDSON, Mott MacDonald Bentley (MMB)

A PETHO, Mott MacDonald Bentley (MMB)

SYNOPSIS

Llyn Celyn is Wales' largest reservoir by volume and is situated approximately 7km north of Bala in North Wales. The reservoir is formed by a rolled clay core supported by gravel-fill shoulders. At 680m wide and 45m high, the reservoir construction was completed in 1966. The dam is a category A dam as defined by Floods and Reservoir Safety, 4th Edition and has a capacity of approximately 81,000,000m³.

Following a statutory inspection Welsh Water was given Measures in the Interests of Safety (MIOS) to ensure that sufficient flood capacity is provided through the spillway system, including a review of the risk of cavitation in the existing spillway and the provision of sufficient discharge capacity to safely pass the Probable Maximum Flood (PMF). The focus of this paper is on the assessment and increase in spillway capacity from 290m³/s to 520m³/s. The paper demonstrates multiple innovative elements including the largest concrete tipping gates in the United Kingdom; a landscaped spillway with improved visual impact compared with the conventional solution, and design and construction, following an observational approach, of a temporary soil nailed excavation due to proximity to a major highway.

BACKGROUND

The reservoir is owned and operated by Welsh Water, but the water level management and releases are the responsibility of Natural Resources Wales (NRW) as part of the Dee Regulation Scheme. Key users of the water downstream include the national whitewater rafting centre (Canolfan Tryweryn), Canal and Rivers Trust, and a number of major water companies including Welsh Water. For completeness, it is noted that statutory recommendations were also made to ensure that all valves are operable. Major pipe and valve works were covered in an earlier paper by Cornelius & McAree (2024) but are summarised in brief here:

Protecting Wales' largest reservoir: dam safety and lasting resilience at Llyn Celyn

- Installing complex temporary upstream isolations designed by MMB's framework diving contractor using inflatable bungs and steel plates assembled at depths of 30m.
- Replacing the butterfly valve with a 0.9m gate valve, installed using a bespoke lifting gantry and trolley system to transport the valve 300m up the tunnel.
- Replacing 2Nr 1.3m diameter fixed cone valves.
- 4Nr new 1.1m diameter gate valves into the existing drop-shaft spillway to augment emergency drawdown capacity.

MMB dam engineers have the good fortune to sit within a uniquely integrated, in-house, UK, design and build team with the dual benefits of a quality-focussed contractor and international expertise within the wider Mott MacDonald Group. This, as well as working under an NEC Option C contract, improved collaboration with the multi-disciplinary project environment. Seven out of 13 subcontractors were local to the scheme and the sound of Welsh language was predominant on site while expertise were sought from across the United Kingdom with reinforcement drawings drafted in Mumbai.

CAVITATION ASSESSMENT OF THE EXISTING DROP SHAFT SPILLWAY

The existing spillway at Llyn Celyn consists of a bellmouth and drop shaft that discharges into a tunnel and stilling pool. The bellmouth inlet comprises reinforced precast blockwork and cast in-situ unreinforced concrete elements. At the bottom of the shaft there is a 90° bend at the upstream end of the tunnel. The tunnel passes through the right abutment of the dam. The vertical shaft and horseshoe shaped tunnel are lined with unreinforced concrete. A cavitation assessment was undertaken for the existing drop shaft spillway. First a visual inspection of the existing spillway was carried out by a concrete specialist followed by a desk-based study reviewing historical physical and computational fluid dynamic modelling reports and analysing the following:

- Cavitation parameter, σ , as defined in Falvey (1990)
- Duration of cavitation
- Aeration at point of cavitation
- Geology
- Structural integrity

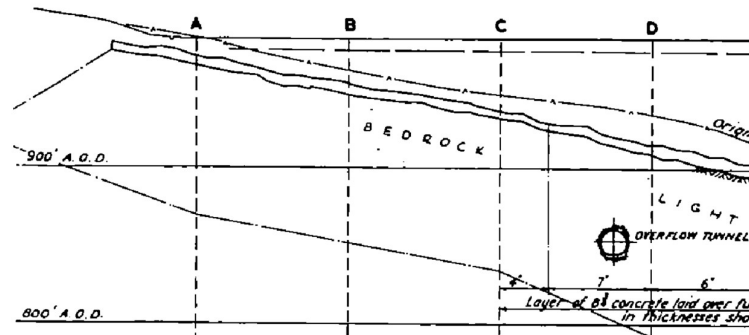


Figure 1. Dam long section at overflow tunnel location from the record drawings, Binnie & Partners, 1966

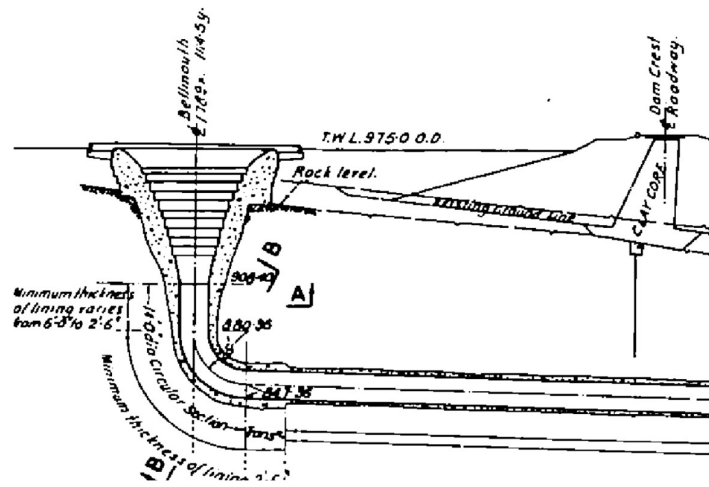


Figure 2. Existing spillway long-section, upstream end from the record drawings, Binnie & Partners, 1966

These factors could then be combined to determine whether damage would be expected to occur during the PMF. The study concluded that no works were necessary at the existing spillway: a successful application of the health and safety and carbon hierarchy of solutions (build nothing).

The remainder of this paper focusses on the new auxiliary spillway.

Protecting Wales' largest reservoir: dam safety and lasting resilience at Llyn Celyn

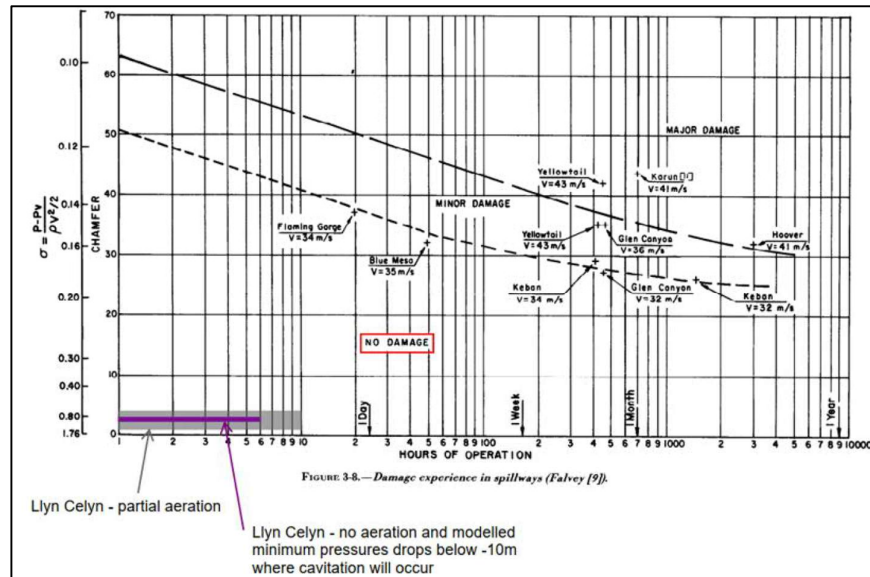


Figure 3. Cavitation at Llyn Celyn existing drop shaft spillway plotted within the “no damage” zone. Original graph from Falvey (1990) annotated by MMB for MMB (2022).

HYDRAULIC MODELLING OF THE NEW AUXILIARY SPILLWAY

The concept design of the new auxiliary spillway is shown in Figure 4 below.

A physical model of the new auxiliary spillway was commissioned by MMB and carried out by CRM Rainwater Drainage Consultancy Ltd. The model was used to optimise the spillway arrangement and directly resulted in a reduction in spillway width at the downstream end with associated cost and carbon savings. Iterations of the physical model also allowed for a flow deflection detail designed to better control the direction of the outflow as demonstrated by the model operator in Figure 6 below. In Figure 6 the painted outer walls represent the initial concept and the unpainted inner walls represent the efficiency achieved through the physical modelling.

Physical modelling results for both spillways were used to create rating curves for a flood study (as well as to review measured pressures at the 90-degree bend) which showed that at the peak of the PMF the flow split is approximately 290m³/s via the existing drop shaft and 230m³/s through the new auxiliary spillway. The PMF could now be safely passed with the proposed new arrangement.

Bulking was estimated using Douma (1943). A comparison was made with other methods, but due to the very shallow slopes of this spillway (maximum slope 1:16.5 V:H) bulked depths were found not to be highly sensitive to the bulking method selected.

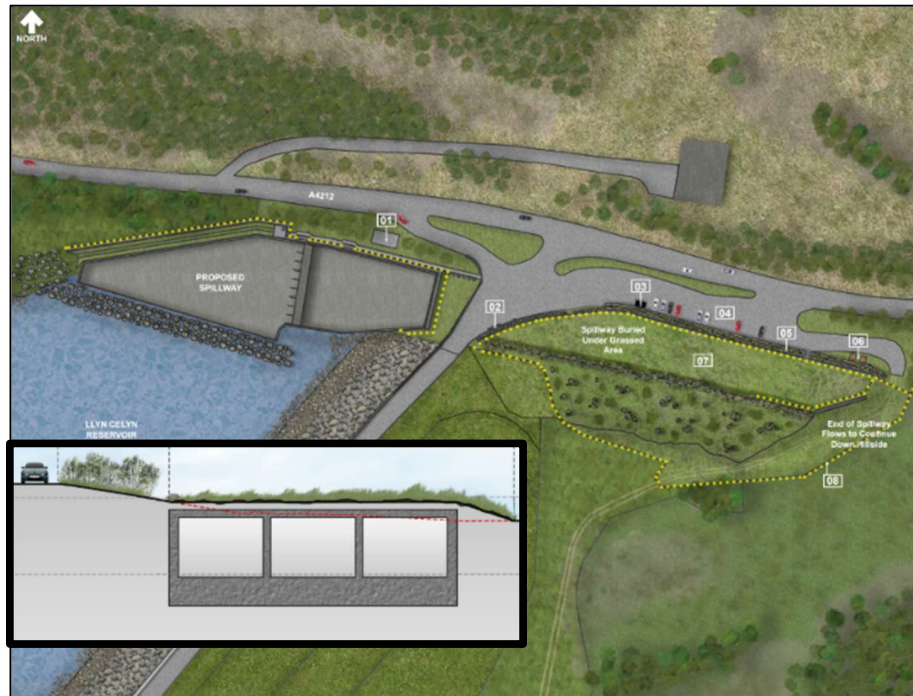


Figure 4. Planning design of the new auxiliary spillway showing the sensitively landscaped culverted section (downstream) and the tipping gates in the upstream tumblebay



Figure 5. Existing spillway physical model (90-degree bend)

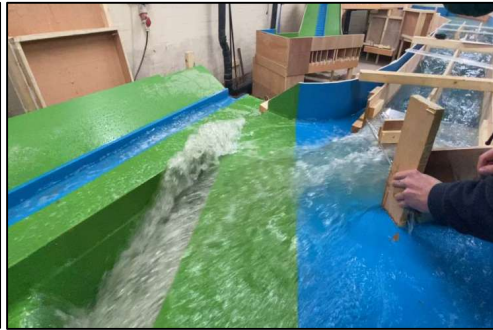


Figure 6. New spillway physical model (outlet)

The new spillway is designed to operate in events of a return period in excess of 10,000 years in order to limit downstream flooding during lower return period events. Thus, the conventional chute from the dam abutment down to the bottom of the valley and river bed was able to be omitted from the design. Also omitting extensive erosion protection between the spillway outlet on the hillside and the river in the valley bottom, MMB used 2D modelling in TUFLOW to confirm the flow path and velocities of the water during emergency operation. The results confirmed that the water would

Protecting Wales' largest reservoir: dam safety and lasting resilience at Llyn Celyn

not be expected to cause flooding to buildings, public infrastructure, or to the embankment. Although velocities would be high enough to scour away some of the soil, any scouring action would provide a channel for the water to flow in which would likely reduce the flow spread which is conservatively not allowed for in the modelling. The worst-case scour is expected immediately downstream of the spillway where the velocities are highest. At this location, an assessment was made to determine an appropriate cut off depth to ensure the integrity of the structure during a PMF. Initial concepts included the option of piling works or material bunds downstream to limit the scour extents but the hydraulic modelling provided sufficient confidence to progress with no further works downstream of the spillway. An extract of that modelling is shown in Figures 7 and 8 below.

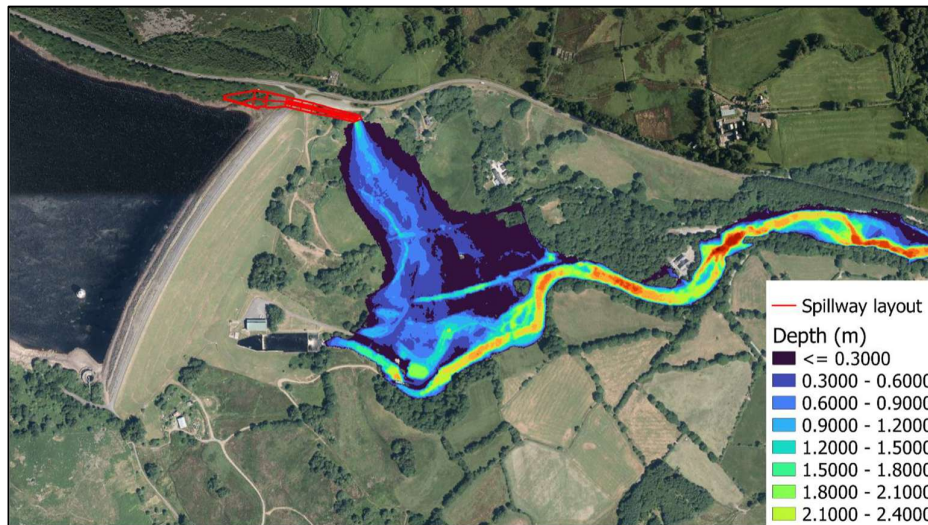


Figure 7. Depth mapping from TUFLOW downstream of the spillway from MMB (2022)

MMB also carried out modelling to compare the downstream flood extents during only the primary spillway operation compared with operating both spillways. The study found that, for the design flood event, the maximum extents downstream would be similar despite the significant increase in flow. Refer to Figure 8 below.

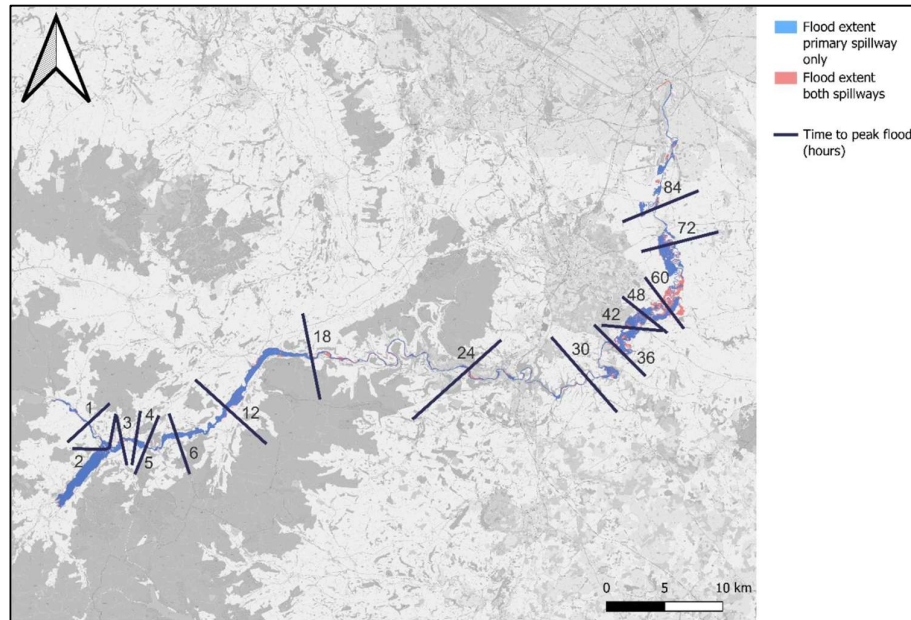


Figure 8. Extract from the downstream modelling from MMB (2025)

SOIL NAILING THE TEMPORARY EXCAVATION

The project team faced a significant constraint for the design of the temporary excavation due to the proximity of the A4212 national highway, with road closures not being permitted. To provide sufficient working area to facilitate construction, the team reviewed various temporary works solutions (e.g. secant piled wall), and selected a soil nailed slope as the preferred method, with a batter angle of 70° from horizontal. The excavation was 7m high x 270m long through the left abutment of the dam. Ground Investigation (GI), comprising both intrusive investigations and non-intrusive geophysical surveys, was undertaken to confirm the ground conditions and inform the design options. The GI proved predominantly granular superficial deposits (sand with a high cobble/boulder content) with lesser cohesive interlayers. This was underlain by the very strong Aran Fawddwy Tuff formation.

Leapfrog Works 3D ground modelling software was used to develop a comprehensive ground model incorporating a digital reconstruction of the embankment dam from archive 2D as-built record drawings (Figure 9). This allowed visualisation of the interaction between the proposed spillway, the temporary excavation, the existing embankment and the interpreted ground conditions. Significant outcomes from this included: a) selection of soil nail stabilisation as the preferred temporary works method to facilitate the

Protecting Wales' largest reservoir: dam safety and lasting resilience at Llyn Celyn

required steep batter angle and; b) the founding depth of the spillway being raised to minimise hard rock excavation.

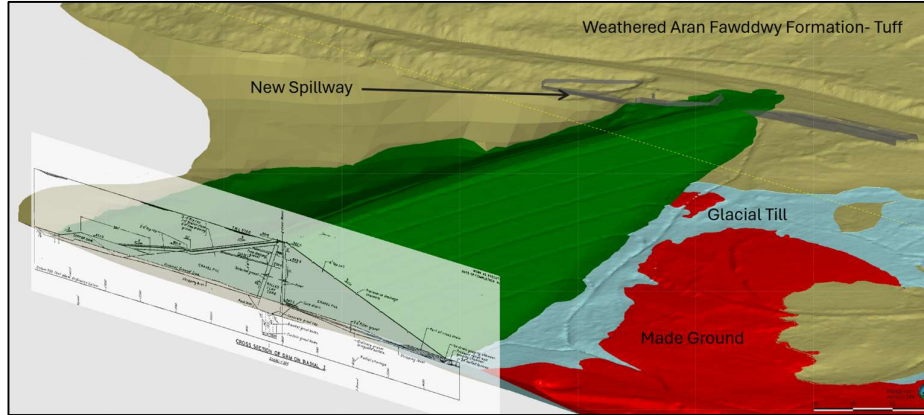


Figure 9. Extract from Leapfrog Works 3D Ground Model.

The outline soil nail design, which was fully compliant with current UK design codes, comprised the following:

- 32mm diameter solid bar nails;
- rigid facing design comprising 200mm thick shotcrete;
- 500mm square face plates, each weighing 59kg, posing a health and safety hazard in respect of manual handling and people-plant interfaces; these plates were later designed out.

The design was subsequently optimised through collaboration with the installation subcontractor (Colin Jones Rock Engineering Ltd). The trial compared on-site ground conditions with those modelled and the suitability of different nail types resulting in: a change to 25mm hollow bar nails; removal of shotcrete facing that was replaced with Geobrugg's Tecco flexible mesh facing system with smaller P33 spike plates, each weighing 2kg and; a reduction in the total number and length of soil nails. The fully excavated trial panel is shown in Figure 10 below.



Figure 10. Soil nail trial panel

Considering this was temporary works for a period of 12-18 months, MMB challenged industry design standard BS 8006 which does not permit the use of flexible facing systems on slopes greater than 60°. Following review of the alternative design proposed by the installation subcontractor using Geobrugg's proprietary design tools (not BS8006 compliant) the lower cost, lower carbon and safer system, from the perspective of health and safety of site installation teams, alternative face support system was accepted.

During site installation benefits were realised through fulltime geotechnical supervision of the works. Geological logging of every nail position was undertaken and fed back into the 3D geological model allowing immediate 'live' updates of the model and understanding of 'true' ground conditions (Figure 11). As updates progressed, it became clear bedrock was 2.4m shallower than the disparate GI inferred. Utilising a prognosis tool built into Leapfrog Works ground modelling software permitted the team to predict ahead geological conditions and reduce both the number and length of nails installed. This resulted in significant time and cost savings; all based on live model updates - the observational approach. The completed soil nail wall is shown in Figure 12.

Protecting Wales' largest reservoir: dam safety and lasting resilience at Llyn Celyn

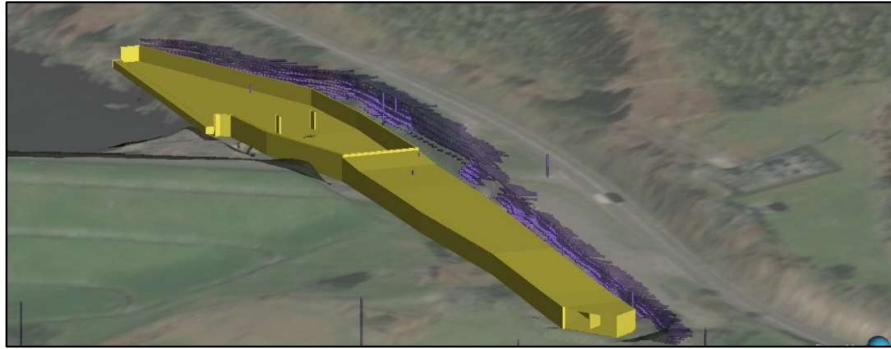


Figure 11. Extract from 3D ground model illustrating all soil nail positions

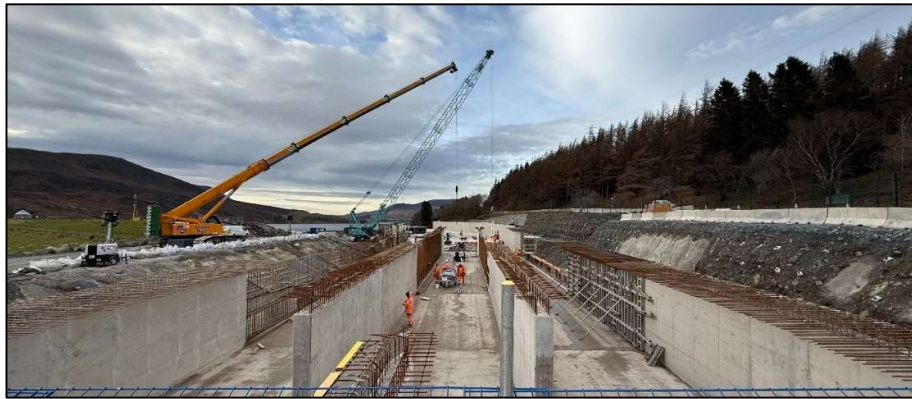


Figure 12. Completed soil nailed slope (right of photograph)

The use of Leapfrog Works had further significant impact on risk reduction using the discrete colour map function to create a traffic light system to enhance the geotechnical analysis of soil nail displacement monitoring data. Monitoring was established post-construction to ensure site operatives and road users' safety. The monitoring regime comprised: surveying vertical groups of soil nails at 10m intervals; establishing trigger levels based on movement and subsequent remedial actions, which were agreed with the client. This allowed live visualisation of the monitoring data and showed that movements above trigger levels were limited to isolated nails with minimal movement observed in neighbouring nails.

Geotechnical design optimisations corresponded to an estimated £1.75M cost saving, compared to the initial tendered sub-contract estimate, and 19,000kgCO₂eq carbon saving.

Although temporary works, because they were not relied upon in the permanent case, the soil nail wall was left in place and backfilled, which avoided additional works to remove the nails. This did not represent any detriment to the permanent works solution or reservoir safety.

TIPPING GATES

The new 33m-wide, 4m-high (maxima) auxiliary spillway has been designed to convey flood events up to the PMF but will only operate in return periods in excess of 10,000 years as mentioned previously. A UK-first installation of reinforced concrete tipping gates with an inclined length in excess of 6m reduced the spillway and weir length by approximately 70%, improving hydraulic performance and delivering major savings. The gates would operate without manual intervention, supporting successful and reliable long-term operation.



Figure 13. Tipping gates viewed from downstream



Figure 14. Tipping gates viewed from upstream during commissioning

Global stability calculations were carried out using conventional mechanics for overturning to design the gate thickness, length and angle. The initial concept was based on other similar, albeit smaller, schemes as documented in Ackers & Hughes (1988) and in Ackers & Scott (2000). Refinements were applied to the methods to account for overturning about the downstream toe of the gate rather than treating the gate as an infinitely thin plate as well

Protecting Wales' largest reservoir: dam safety and lasting resilience at Llyn Celyn

as sensitivity checks to account for the non-rectangular final elevational shape and other variables such as the density of reinforced concrete and construction tolerances.

A supporting truss has been installed to support the top edge of the tipping gates. A finite element model was carried out to optimise the design of the metalwork truss and piers, and supporting concrete structure. The 3D visualisation used as an input to the finite element analysis (FEA) is shown in Figure 15 below.

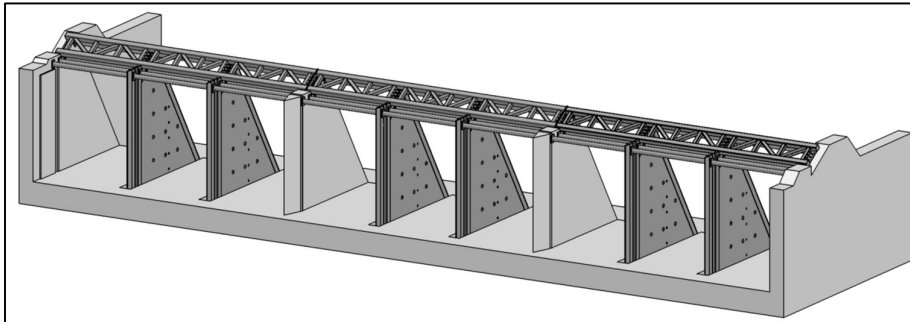


Figure 15. 3D model of the tipping gate supports for FEA

Commissioning was successfully carried out by pumping water between a stoplog arrangement and the gates on the upstream side and measuring the depth of water required for the gate to undergo “initial movement” and then “significant leakage”. Once the gate started to move from the truss, and significant leakage occurred, the water level rapidly reduced, and the gate returned to its original position. In the unlikely event that the gate would continue to tip, the crane that was used to install the gates was used to hold the top of the gate as shown in Figure 17 below.

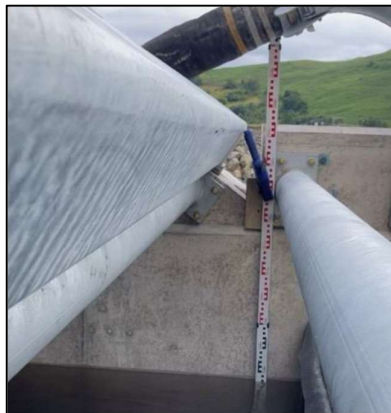


Figure 16. Commissioning the gates: water level measurement upstream of the gate



Figure 17. Commissioning the gates: photograph following a small movement away from the supports

The table below shows the actual trigger level compared with the theoretical trigger level for the three known similar schemes.

Table 1. Data for Greenfield and Yeoman Hay from Ackers & Hughes (1988) and data for Llyn Celyn from this project.

Scheme	Gate	Theoretical depth	Depth at initial movement		Depth at major leakage		
			Depth (m)	Difference (%)	Depth (m)	Difference (%)	Arithmetic Mean
Greenfield	1	1.54	1.53	-1%	1.53	-1%	-2%
Greenfield	2	1.54	1.48	-4%	1.48	-4%	
Greenfield	3	1.54	1.53	-1%	1.53	-1%	
Yeoman Hay	2	1.95	1.71	-12%	1.82	-7%	-6%
Yeoman Hay	3	1.95	1.73	-11%	1.76	-10%	
Yeoman Hay	4	1.95	1.68	-14%	1.86	-5%	
Yeoman Hay	5	1.95	1.71	-12%	1.79	-8%	
Yeoman Hay	6	1.95	1.73	-11%	1.75	-10%	
Yeoman Hay	7	1.95	1.7	-13%	1.78	-9%	
Yeoman Hay	8	1.95	1.73	-11%	1.93	-1%	
Yeoman Hay	9	1.95	1.78	-9%	1.87	-4%	
Yeoman Hay	10	1.95	1.73	-11%	1.83	-6%	
Yeoman Hay	11	1.95	1.9	-3%	1.93	-1%	
Yeoman Hay	12	1.95	1.95	0%	1.95	0%	
Llyn Celyn	1	2.85	2.52	-12%	2.78	-2%	-3%
Llyn Celyn	2	2.85	2.65	-7%	2.77	-3%	
Llyn Celyn	3	2.85	2.7	-5%	2.79	-2%	
Llyn Celyn	4	2.85	2.65	-7%	2.79	-2%	
Llyn Celyn	5	2.85	2.45	-14%	2.69	-6%	
Llyn Celyn	6	2.85	2.52	-12%	2.72	-5%	
Llyn Celyn	7	2.85	2.47	-13%	2.76	-3%	
Llyn Celyn	8	2.85	2.5	-12%	2.7	-5%	
Llyn Celyn	9	2.85	2.75	-4%	2.96	4%	

ENVIRONMENTAL IMPACT ASSESSMENT AND PLANNING APPLICATION

Llyn Celyn is situated in Snowdonia National Park (SNPA) and is surrounded by Sites of Specific Scientific Interest (SSSIs), Special Areas of Conservation (SACs) and Special Protection Areas (SPAs). As part of the planning application to SNPA, an Environmental Impact Assessment (EIA) was required to demonstrate that the project will minimise environmental impacts during and after construction. The planning permission was granted in April 2023 with high compliments from the associated planning consultees.

Releases from Llyn Celyn reservoir play a vital role in the regulation of the River Dee, so these works required extensive collaborative planning with various stakeholders including Natural Resources Wales (NRW) to ensure the works could be safely completed, whilst minimising the risk to water resources. The site falls within the boundaries of Eryri (also known as Snowdonia) National Park and a full Environmental Impact Assessment was carried out by MMB. Planning permission was subsequently granted by the national park authority.

As Llyn Celyn lies within Snowdonia National Park, the spillway has been carefully designed to blend in with its natural surroundings and work with the land adjacent to the dam. All exposed concrete walls have either a

Protecting Wales' largest reservoir: dam safety and lasting resilience at Llyn Celyn

patterned concrete finish or stone cladding. Following a successful trial panel – the project team challenged the planning design to gain planning approval to reduce the stone cladding by 75% and replace it with the patterned concrete finish. The patterned finish was carefully selected to match the stonework to the existing structures on site.

THE MAIN SPILLWAY STRUCTURE

The spillway structure is cast in-situ reinforced concrete anchored into mass concrete beneath the base.

The spillway is designed as 18 no. separate bays with free movement joints provided between each bay. Bays 1 to 8 are wide channels formed from in-situ reinforced concrete base and side walls. Bays 9 to 18 are culverts formed from in-situ reinforced concrete base, roof, side and inner walls. The reinforced concrete base for all the bays is dowelled into a mass concrete base that is utilised for global stability. Bay 6 incorporates internal reinforced concrete and steel piers that support a steel roof truss that in turn supports the head of the precast concrete tipping gates.

To improve global stability, reinforced concrete heels have been added as required to one or both sides of the spillway bays. The heels have been introduced to mobilise the beneficial vertical load of the material on top of the heels.

The global stability resistance relies on the reinforced concrete base being connected to the underlying mass concrete. The dowels between the reinforced and mass concrete slabs are designed to transfer the flotation and sliding forces calculated from the global stability checks.



Figure 18. Enabling works from the hydraulic right side of the valley



Figure 19. Enabling works at the new spillway location

In UK spillway terms the volumes of concrete poured were significant (circa 12,000m³). On site, an open-plan office with collaborative planning boards encouraged engagement between management, engineers, labour and

subcontractors - vital to effectively plan concrete pours, given the scale and volume of concrete required. Due to the site's remote location, this demanded close collaboration with the supplier – to assess frequency and quantity of concrete required and manage deliveries.

The original programme for the spillway concrete structure was 78 weeks from early November 2023 to end of April 2025. MMB was able to work closely with subcontractors CSF and Cambrian-UK to deliver the spillway concrete elements in 65 weeks. This resulted in a betterment when compared to the original programme dates, even with a delayed start to the project. With detailed collaborative planning the project team was able to overcome key risk elements such as weather and concrete related issues due to the remote geographical location. This resulted in an estimated cost saving of over £0.5m for the concrete structure.

The concrete designed for this scheme allowed for a key risk element which was the remote location of the site, approximately one hour away from the nearest batching plant. The mix designs were developed with the concrete supplier (Cambrian-UK), including a pre-commencement trial to ensure suitability in terms of meeting the structural requirements and constructability on site. The structure consisted of two different mixes – C16/20 for the mass fill foundation and C40/50 for the structure base slabs, walls and roof. Each mix consisted of a 50/50 cement-GGBS blend with ADVA635 water reducing admixture. The C40/50 mix included the Sika Tard-R which significantly improved retaining workability.

During pre-construction stage a maximum concrete pour volume of 150m³ - 200m³ was assumed due to batching plant location, logistics and construction site restraints. Through project experience and collaboration with suppliers the project team was able to gain confidence in pouring up to 350m³ per day and maintain pouring concrete on consecutive days with some weeks exceeding 1000m³.

The structure was constructed in five phases: the base slabs and walls of the downstream side of the dam were constructed first followed by the upstream tumblebay and then finally the section through the dam crest. This provided a protective concrete surface upstream and downstream of the dam core should an extreme event occur during Phase 5 (Figure 20).

Protecting Wales' largest reservoir: dam safety and lasting resilience at Llyn Celyn

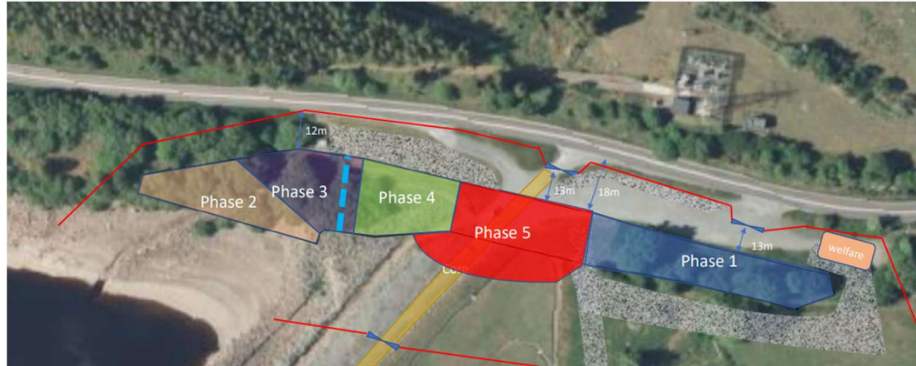


Figure 20. Construction phases

The initial proposal, for tying the spillway into the impermeable dam core, was reinstatement of the existing core and downstream filter tied in with a concrete irregular pyramidal frustum. This has the benefits of elongating the seepage path and the pyramidal shape provides a sloped angle for sound compaction of the backfill material. The clay core, where exposed in the soil nailed wall to the temporary excavation, was protected by shotcreting while the construction of the spillway was ongoing. Due to the steeper excavation face (70°) giving rise to a relatively narrow and steep geometry on the left-hand side a purely earthworks tie-in was not considered desirable due to increased risk of arching action and differential shears and cracks. Therefore, the design team developed a concrete cut-off wall solution to extend into the left-hand excavation face and abutment. A steep trench would need to be cut into the soil nailed excavation for the concrete wall to tie into with challenging angles of both the 70° from the vertical as well as a 60° horizontal interface between the cut-off wall and the excavation. Due to concerns around complex temporary works to support a steel reinforcement cage with the unusual geometry and in order to limit the time of exposure of the clay, fibre-reinforced concrete was utilised for the cut-off wall. To further reduce risk of excessive seepage through joints and interfaces, the as-dug cohesive fill was used to backfill against the concrete cut-off wall both upstream and downstream.

As part of the construction of the spillway the piped River Hesgyn indirect catchment had to be diverted around the new spillway. At the downstream (reservoir) end a multi-stage flow measurement weir was constructed in a concrete chamber. The maximum flow rate of the indirect catchment is 2.4m³/s so the diversion of the pipework and concrete culverts was a significant undertaking. The replacement pipework is ductile iron and the spigot-socket joints were fully encased with fibre-reinforced concrete within

the clay core, and this concrete surround was integrated with the construction of cut-off wall referred to above.

Excavating through the dam crest and constructing that phase of the new spillway was a safety-critical activity requiring extensive planning due to the operational demands put on the reservoir and large catchment area, managed under DCWW Silver Command.

The dam safety construction management plan linked the construction programme to the reservoir water level management plan maintained by Natural Resources Wales (NRW). It was crucial to maintain programme during these works and the team leveraged expertise from earlier phases of the scheme, reducing construction risks, costs, and programme duration. A 'temporary drawdown curve' was carefully agreed with the stakeholders to strike a careful balance between flood risk mitigation, water resource, and river regulation requirements. Programme certainty for this element was crucial, and the schedule was fine-tuned through combined expertise across all delivery teams.

The large-scale nature of this scheme, and its location in a sensitive setting, naturally involved multiple stakeholders, so transparent and regular communication was of high priority. Frequent meetings with Natural Resource Wales (NRW) facilitated open consultations on flow modifications, reservoir drawdowns and other interventions which could affect downstream stakeholders. Updates were then shared with the Dee Consultative Committee (DCC), comprising downstream water users like Canal and River Trust, DCWW, Hafren Dyfrdwy, United Utilities, and Natural Resources Wales (NRW). Other stakeholders included Gwynedd Council, Snowdonia National Park Authority, Canolfan Tryweryn, and the public.

Carbon savings have been discussed throughout this paper. Other sources of savings included:

- The use of intelligent machine control to minimise waste
- Fibre reinforced concrete where possible (e.g. left hand cut-off wall)
- Binder replacement such as GGBS for concrete
- Formlined walls as a saving against transporting stone masonry long distances
- All of the soil excavated on site has been reused

Protecting Wales' largest reservoir: dam safety and lasting resilience at Llyn Celyn



Figure 21. Reinforced concrete bases under construction upstream and downstream of the dam core



Figure 22. Site team photograph during construction

CONCLUSIONS

Remedial works at Llyn Celyn have been undertaken to protect Wales' largest reservoir providing dam safety and lasting resilience. The tipping gates had not been attempted on this scale before and this paper brings together tipping gate testing data from three different dams. A theme running through the project is challenging the scope to provide a proportionate and cost-effective solution. A mix of 2D (computational) and 3D (physical) hydraulic modelling has been used to optimise the solution. The temporary soil nailing utilised the observational approach integrated with 3D ground modelling delivering substantial commercial and carbon

efficiencies. A truly integrated design and build project team, with close collaboration with the project stakeholders and the supply chain, has delivered a successful and multi-award-winning project.

ACKNOWLEDGEMENTS

The client for the scheme was Dŵr Cymru Welsh Water, and the Principal Designer and Principal Contractor were Mott MacDonald Bentley (MMB). Other significant contributions included from Ant Hire, AJ Butler, Breedon Aggregates, Celtic Masonry, Cambrian, Celtest, Colin Jones Rock Engineering, Concrete Structures & Floors, PAM Saint Gobain, CRM Rainwater Drainage Consultancy, Mona Lifting, and RAM Services. The authors' sincere thanks go to all involved in the project.

The authors also wish to thank Dŵr Cymru Welsh Water for permission to publish this paper.

REFERENCES

- Ackers JC and Hughes AK (1988) Tipping Gates for Auxiliary Spillway Control. In *Proceedings of a symposium by the British National Committee on Large Dams, University of Manchester on 14-17 September 1988*. BNCOLD, pp. 251-254.
- Ackers JC and Scott CW (2000) Dam innovation – a selection of innovative features and curiosities. In *Proceedings of the biennial conference of the BDS held at the University of Bath on 14-17 June 2000*. Thomas Telford, London, UK, pp. 64-71.
- Cornelius G and McAree M J (2024) Case studies from challenging pipes and valves works. In *Managing Risks for Dams and Reservoirs - Proceedings of the 22nd British Dam Society Conference* (Thompson A and Pepper A (Eds)). British Dam Society, London, UK.
- Douma, J. H. (1943) *Proceedings of the American Society of Civil Engineers; Volume 69; No. 8; Part 2; Transactions Number 108*, pp. 1462-1473.
- Falvey, H. (1990) *Cavitation in chutes and spillways; Engineering Monograph no. 42*. USBR.
- Institution of Civil Engineers (2015) *Floods and Reservoir Safety; Fourth Edition*.
- Mott MacDonald Bentley (17/10/2022) *Llyn Celyn Section 10 MITIOS Works; Existing Overflow – Cavitation Assessment*.
- Mott MacDonald Bentley (06/06/2025) *Llyn Celyn Section 10 MITIOS Works; Downstream Extent Model*



The British Dam Society

Townhead Reservoir – The Real-World Impact of Changes to UK Flood Hydrology

JENNIFER HOLLOWAY, Mott MacDonald

ALASDAIR WALKER, Mott MacDonald

PHOEBE ERSKINE, Mott MacDonald

SYNOPSIS

Hydrology in the UK is regularly updated with more recent rainfall data and improved rainfall-runoff model generation methods. The FEH22 rainfall data set was released in December 2022 and the latest calibration of the ReFH2.3 model was released in 2023 to include the FEH22 rainfall. In September 2025, the Flood Estimation Guidelines were updated by the Environment Agency with a considerable update to the flood estimation for reservoir safety section (as recommended by Fraser (2024)). It is now recommended that the FEH22 dataset is used for all return period events in conjunction with the ReFH2.3 runoff model.

The latest S47 inspection of Townhead Reservoir made it a Measure in the Interests of Safety to investigate and repair the 27m wide auxiliary spillway. Following the investigations, it was concluded that the auxiliary spillway was beyond economic repair and that the construction of a new off-dam auxiliary spillway was the best option. The continued need for the auxiliary spillway was based on a 2011 flood study which used FEH99 catchment descriptors, and FSR rainfall and rainfall-runoff methodology. An updated flood model, using the latest guidance, was constructed to assist in the design of the new spillway, but this revealed that the auxiliary spillway was not in fact needed to pass flood flows.

This paper will provide a case study of the history of the hydrology and flood routing undertaken and demonstrate the real-world impact that the changes on hydrology have had on this reservoir.

INTRODUCTION

Hydrological modelling guidance in the UK is regularly updated to include contemporary rainfall data and improvements to the runoff model generation methods. The FEH22 rainfall data set was released in December 2022 and the latest calibration of the ReFH2.3 model was released in 2023 to incorporate the FEH22 rainfall data set. In September 2025, the Flood Estimation Guidelines were updated by the Environment Agency with a

Townhead Reservoir – The Real-World Impact of Changes to UK Flood Hydrology

considerable update to the flood estimation for reservoir safety section (as recommended by Fraser {2024}).

Townhead Reservoir is used as a case study to demonstrate the impacts of these changes over time. Flood studies have been undertaken for Townhead in 1999 and 2011, with the most recent being completed by Mott MacDonald in 2025. The rainfall datasets and runoff models used form the basis of these studies, therefore informing critical design decisions made to ensure the safety of dams in the UK.

This paper will provide a case study of the history of the hydrology and flood routing undertaken and demonstrate the real-world impact that the changes on hydrology have had on this reservoir.

TIMELINE OF UK FLOOD HYDROLOGY

A detailed timeline of UK flood hydrology for UK reservoirs is given by Fraser (2024). Detailed timelines for rainfall and runoff models are presented separately for 1-in-150-year, 1-in-1,000-year, 1-in-10,000-year and Probable Maximum Flood (PMF) events. For this paper, a combined timeline has been reproduced in Figure 1 for 1-in-10,000-year and PMF events only, detailing the combinations of rainfall and runoff models recommended in various guidance documents for use in reservoir flood studies.

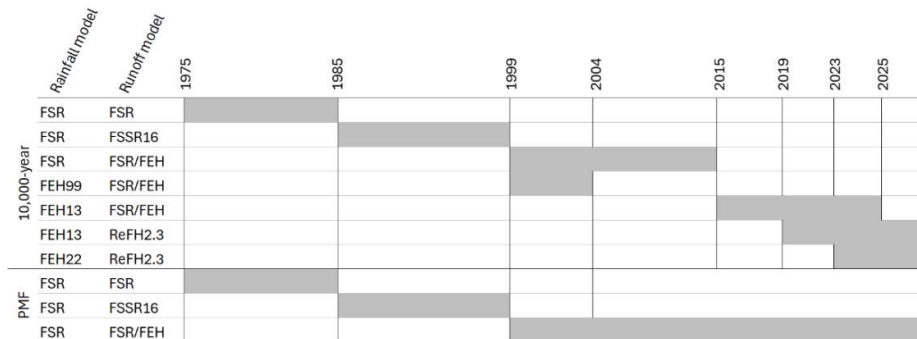


Figure 1. Timeline of rainfall and runoff models recommended for UK reservoir flood studies for 1-in-10,000-year and PMF events

CASE STUDY – TOWNHEAD RESERVOIR

Background

Townhead Reservoir was built east of Kilsyth in the early 1770s as a feeder for the Forth and Clyde canal, a function which it serves to this day. The reservoir was formed with the construction of a 7m high earth embankment dam over the Banton Burn, an outlet from the small Banton Loch. This embankment incorporates a low-level outlet through a masonry culvert

which allows flows to be released to the canal. The reservoir's original spillway was constructed remote from the embankment through a low spot in the reservoir rim and has a formalised concrete spillweir discharging to a rough rock channel.



Figure 2. Main Concrete Weir



Figure 3. Main Spillway Channel



Figure 4. Simple Map of Townhead Reservoir (AKA Banton Loch)

The embankment is fairly typical of the canal reservoirs built at that time, with steep upstream and downstream faces, 1(V):2(H) and 1(V):1.6(H)

Townhead Reservoir – The Real-World Impact of Changes to UK Flood Hydrology

respectively and a relatively wide crest of 4.5m. There are no drawings of the embankment, so until recently the internal make-up of the embankment was unknown, but a recent ground investigation has identified a roughly centralised clay core.



Figure 5. Crest and Downstream Face of Embankment

The capacity of the reservoir is approximately $817,100\text{m}^3$ and the surface area at its top water level of 67.19mAOD is 0.26km^2 .

In the mid-1930s, in response to the Reservoirs (Safety Provisions) Act 1930, a substantial auxiliary spillway was added to the reservoir. The approximately 27m wide spillweir was constructed within the crest of the embankment, 400mm higher than top water level. Below the spillweir, the spillway was formed from a series of concrete steps leading to the downstream toe of the embankment.



Figure 6. Auxiliary Spillway

There is no design report to support the 1930s work, and there is no indication of what flows the auxiliary spillway was designed for, but flood studies were conducted in both 1999 and 2011 and these studies demonstrated that the auxiliary spillway was essential for safely conveying the 1-in-10,000-year flood event, the Safety Check Flood for a Category B reservoir.

By the late 2010s the auxiliary spillway was in very poor condition despite several rounds of major repairs and there was increased focus on addressing the long-standing condition concerns for this essential asset following the 2019 Toddbrook incident.

The 2019 inspection under Section 47 of the Reservoirs (Scotland) Act 2011 by Robert J Mann of AECOM made it a Measure in the Interests of Safety (MIOS) to:

“Investigate the possibility of a gap or voids beneath the floor and side walls of the auxiliary spillway by drilling, water testing and carrying out grouting or other measures as deemed appropriate by the appointed qualified engineer. Include for investigation and any repair deemed appropriate of the defects in the shotcrete surfacing to the cascade.”

Following the investigation phase of the MIOS, it was concluded that the auxiliary spillway was beyond economic repair and that the construction of a new off-dam auxiliary overflow was the best option to resolve the MIOS.

Townhead Reservoir – The Real-World Impact of Changes to UK Flood Hydrology

In early 2024, Scottish Canals contracted Mackenzie Construction Ltd to undertake the design and construction of the new spillway, and Mackenzie Construction Ltd sub-contracted the design services to Mott MacDonald.

At the outset of the design phase, the new spillweir was envisaged as being approximately 27m long to match the length of the existing auxiliary spillweir. As one of the first design tasks, Mott MacDonald re-ran the flood routing to determine the length of new auxiliary spillweir which would be required with an ogee weir (C_d of 2.1) compared to the existing auxiliary (C_d of ~1.4).

Previous Flood Studies

Prior to the work associated with this design, the latest flood study was carried out in 2011. It assessed the 1-in-10,000-year flood event, using the rainfall from the 1975 Flood Study Report (FSR) in conjunction with the Flood Study Report / Flood Estimate Handbook (FEH) runoff model. The principal results from the flood study are replicated in Table 1.

Table 1. Summary of 2011 Flood Study Results

Key Outputs	2011 Results
Rainfall Depth	185mm
Peak Inflow	59.99m ³ /s (inc. 6.0m ³ /s from indirect catchment via Colzium Cut)
Peak Outflow (Main Weir)	29.4m ³ /s
Peak Outflow (Auxiliary Weir)	25.0m ³ /s
Peak Outflow (Total)	54.4m ³ /s
Peak Stillwater Flood Level*	68.33mAOD

*Corrected to TWL=67.19mAOD

The flood study prior to the 2011 study was undertaken in 1999 and therefore, utilised the FSR rainfall and FSR runoff model. Both flood studies indicated that during a 1-in-10,000-year flood event, a substantial depth of flow would pass over the auxiliary spillweir, with the 2011 study indicating a spill depth of approximately 700mm.

Hydrological and hydraulic analysis

The two Townhead Reservoir embankments were designated as Category B for flood protection in the 2019 S47 Inspection Report. In accordance with Flood and Reservoir Safety 4th Edition (FRS4), Table 2, the inflow requirements are, therefore, a 1-in-1,000-year flood event and 1-in-10,000-year flood event for the Design Flood and the Safety Check Flood respectively. As the Safety Check Flood is the more onerous event, it was used to verify the hydraulics of the existing auxiliary spillweir and confirm

the length of new auxiliary spillweir which would be required with an ogee weir. This initial design work also included an update to the hydrological and hydraulic analysis using contemporary methods.

Rainfall and Runoff Models

As the previous flood study pre-dates the release of both the FEH13 and FEH22 depth-duration-frequency models (DDFs), assessments using both data sets were undertaken for comparison with the previous results. For a 3-hour duration, 1-in-10,000-year storm, the rainfall depths were 128.8mm and 98.73mm in FEH13 and FEH22 datasets, respectively. Both rainfall datasets were assessed for comparison and to obtain an understanding of any residual uncertainties there might have been with the model.

At the time of the 2025 work, the Flood Estimation Guidelines (Environment Agency, 2025) had not yet been updated to reflect the suggested method given by Fraser (2024). Although the ReFH2.3 runoff model had been extended and calibrated for the 1-in-10,000-year event, it had not yet been recommended for use in UK reservoir flood studies. Both the FEH and ReFH2.3 runoff models were both therefore, assessed for comparison in this study, with the ReFH2.3 model results being used as a sensitivity check.

The FEH13 rainfall was initially adopted as the higher rainfall depth between the two most recent datasets, to allow for some redundancy in the design of the new auxiliary spillway. The initial assessment therefore used the FEH13 DDF in both runoff models for comparison.

The FEH22 DDF is based on a substantially larger dataset than the FEH13 DDF, including more rain gauges with hourly rainfall data, as well as more storms with shorter durations. The critical storm duration for Townhead Reservoir was found to be 8.7 hours. As the storm duration is below 12 hours, it was considered more appropriate to use the FEH22 DDF for the assessment. It was also considered overly conservative to adopt the FEH13 rainfall depths for redundancy in the spillway design due to the substantial difference in the rainfall depths and the increased accuracy of the FEH22 dataset at this storm duration.

As a sensitivity check the FEH22 data was run using the ReFH2.3 model, with the peak flood rise from the ReFH2.3 model run only being 0.01m greater than the FEH model run.

The final results adopted for the study were from the FEH22 DDF in combination with the FEH runoff model.

Townhead Reservoir – The Real-World Impact of Changes to UK Flood Hydrology

Wave Overtopping

Given that this analysis was being undertaken for design purposes, rather than assessment purposes, the maximum stillwater flood rise which resulted in an acceptable wave overtopping was first calculated, as this would allow the spillweir to be more accurately sized.

The significant wave height within the reservoir was calculated using the Donelan / JONSWAP method set out in FRS4, and the wave overtopping was calculated using the methods set out in EurOtop II (2015).

In accordance with FRS4, Table 6.2, the allowable limit for mean wave overtopping during the Safety Check Flood was set at 1 litre per second per metre (1l/s/m) as the crest and downstream face of the embankment was of good grass-covered clay fill. The starting level for the assessment was then varied to match the assessed overtopping to the 1l/s/m acceptable overtopping. The starting level which resulted in 1l/s/m of wave overtopping then became the target peak stillwater flood level for the flood routing. This target level included an allowance for future settlement of 100mm.

Flood Routing

Flood routing was undertaken using Flood Modeller software. Water levels at the beginning of the simulation were set to the TWL of the reservoir, in accordance with Table 2.1 of FRS4. Two outlets were modelled from the reservoir representing the main and auxiliary overflows. These were simply modelled as spill units with no representation of the downstream channel, due to the study assessing the overtopping and weir capacity rather than the spillway channel capacity more generally.

When the 2025 study was undertaken the formal guidance did not yet recommend the use of the ReFH2.3 model for reservoirs and there was negligible difference between the runoff model results when using the FEH22 rainfall, therefore the FEH22 results using the FEH runoff model were presented to the Client.

Table 2. Comparison of 2011 flood study and 2025 sensitivity tests to final 2025 results when the existing arrangement is run

	2011 Results	2025 Results (FEH13 rainfall, FEH runoff model)	2025 Results (FEH22 rainfall, ReFH2.3 runoff model)	2025 Results (FEH22 rainfall, FEH runoff model)
Duration (hrs)	6.44	8.25	8.7	8.7
Peak Inflow (m³/s)	60.0	48.1	43.2	42.5
Peak Outflow (m³/s)	54.4	44.9	35.1	34.6
Rainfall Depth (mm)	185.2	152.2	129.5	129.5
Max. Stage (mAOD)*	68.33	68.22	68.00	67.99
Flood Rise (m)	1.14	1.03	0.81	0.80

**Different runs applied different TWLs, but this table has been standardised using TWL=67.19mAOD*

A C_d of 2.1 was assumed for the new auxiliary spillweir, and a C_d of 1.98 was assumed for the existing main spillweir. The existing auxiliary spillweir has a C_d of ~1.4 confirmed via physical modelling, so the initial run for the new spillweir was an auxiliary two-thirds the length of the existing auxiliary to accommodate the increase in efficiency. Initially, it was assumed that the new auxiliary spillweir would be approximately 18m long, but this returned a peak stillwater flood level well below the target level, and the model was then iterated by adjusting the auxiliary spillweir length to align the peak stillwater level with the target level.

The auxiliary spillweir length was reduced in 2m increments until it was 0m wide i.e. no auxiliary spillway required. This final model run surpassed the target peak stillwater flood level by 70mm, giving a future settlement allowance of only 30mm, but it indicated that the reservoir could pass virtually a 1-in-10,000-year storm event without flow passing over the auxiliary spillweir.

Conclusion of Studies

The 2011 freeboard assessment stated that the existing auxiliary spillway arrangement is insufficient to pass a 1-in-10,000-year flood without exceeding the allowable limit for wave overtopping. Recommendations therefore included lowering the top water level, increasing the spillway

Townhead Reservoir – The Real-World Impact of Changes to UK Flood Hydrology

capacity, or raising both the secondary dam and auxiliary spillway walls. As a result of this study: a small wave wall was added to the lower subsidiary dam; and the footbridge across the main spillway was raised.

In contrast, the 2025 flood study concluded that the auxiliary spillway is not required to pass a 1-in-10,000-year flood, with the main spillway providing sufficient capacity on the condition that the downstream channel be formalised and the freeboard be enhanced to provide future settlement allowance.

DISCUSSION

It is now recommended in the Flood Estimation Guidelines that the FEH22 rainfall dataset is used for all return period events in conjunction with the ReFH2.3 runoff model. The dramatic change noted above demonstrates the necessity of repeating historic flood studies, or those using historic hydrological guidance and / or data, applying both recent guidance and data sets. This is especially pertinent when flood study results are being used as a basis or justification for major design decisions.

The cost of physical modelling, numerical modelling, and additional flood modelling should always be weighed up against the cost implication of uncertainty on the design being developed. If modelling is selected as being necessary, this process should begin as early into the project as is feasible to confirm any design assumptions and engage consultants to undertake the design phase of the project.

CONCLUSION

Following the investigations, it was concluded that the auxiliary spillway was beyond economic repair and that the construction of a new off-dam auxiliary spillway was the best option. The continued need for the auxiliary spillway was based on a 2011 flood study which used FSR rainfall data and runoff method. An updated flood model, using the latest guidance, was constructed to assist in the design of the new spillway, but this revealed that the auxiliary spillway was not in fact needed to pass flood flows, and that the reservoir could pass the standard set out in FRS4 with only minor modifications. This represented a potential saving to Scottish Canals of in excess of £5M.

REFERENCES

Environment Agency (2025), *Flood Estimation Guidelines (Version 10)*, Environment Agency, Bristol.

- Robert Fraser (2024), An updated reference table for extreme flood hydrology methods at UK dams. *Dams and Reservoirs* **34(4)**: 163–166, <https://doi.org/10.1680/jdare.2024.13>
- Institution of Civil Engineers (2015). *Floods and Reservoir Safety* (4th edn.). ICE, London.
- Van der Meer et al. (2018). *EurOtop – Manual on wave overtopping of sea defences and related structures* (2nd Edn.). www.overtopping-manual.com.



The British Dam Society

Bewl Drawdown Improvements Scheme: Design, Delivery and Commissioning of a Large-Scale Siphon System

PHILIP BARNARD, Stantec
JON TROKE, Stantec
LAURA MANSITO, Southern Water

SYNOPSIS

Bewl Water Reservoir, located within the High Weald Area of Outstanding Natural Beauty near Tonbridge Wells, is the largest inland water body in Southeast England. Since construction completion in 1976, the reservoir has fulfilled a dual function as a critical asset for water supply to Kent and East Sussex while also providing amenity value for a wide range of water sports, outdoor pursuits and wildlife. A statutory Section 10 inspection undertaken in 2018 identified that the existing drawdown infrastructure did not meet current standards for reservoir safety. A recommendation was therefore made to provide enhanced facilities capable of reducing the full supply level (FSL) by five metres within five days, with Q_{50} inflow conditions.

The selected solution comprised three large-diameter siphon pipes over the crest of the embankment dam, supported by an uprating of the existing bottom outlet works. The siphons were designed to deliver a combined discharge capacity of approximately $37.5 \text{ m}^3/\text{s}$, making it one of the largest siphon drawdown systems in the United Kingdom. Construction started in 2023 and the scheme was successfully commissioned in Spring 2026. This paper presents the development of the scheme through design, construction and commissioning.

INTRODUCTION

EA/Defra Guidance published in 2017 and the Toddbrook Incident in 2019 highlight the importance of robust emergency drawdown facilities at UK reservoirs. The ability to rapidly reduce reservoir levels is a critical element of dam safety, providing a means to reduce hydraulic loading during emergency events, facilitate easier and safer interventions and to minimise the consequences of a subsequent breach.

Bewl Water Reservoir is formed by a substantial earth embankment dam approximately 28m in height and 930m in length, impounding a storage volume of around 31 million cubic metres. The reservoir is situated in a

Bewl Drawdown Improvements Scheme: Design, Delivery and Commissioning of a Large-Scale Siphon System

highly sensitive environmental setting and functions not only as a key water supply asset but also as a popular recreational destination. These competing demands introduce significant constraints on any major engineering works.

This paper documents the development and delivery of the Bewl Drawdown Improvements Scheme, providing insight into the engineering and environmental considerations that affected design and construction.

BACKGROUND AND DRIVERS

The primary driver for the scheme originated from a statutory Section 10 inspection in 2018, which noted a shortfall in drawdown capacity against current industry guidance (Defra/EA, 2017) and included a recommendation in the interests of safety to upgrade the drawdown facilities so that the upper 5m of the reservoir could be lowered within a period of 5 days. This is consistent with the guidance that limits the drawdown rate to approximately 1m per day for larger dams.

The development of the improved drawdown system was constrained by several key requirements:

- to maintain the reservoir at normal seasonal water levels throughout construction and to therefore minimise intrusion into the existing dam structure;
- to maintain draw-off flows to the water supply system and provide the required compensation and augmentation flows to the River Bewl throughout construction and commissioning;
- to comply with stringent environmental protection requirements;
- to enable safe routine siphon wet testing without harming the downstream watercourse, in accordance with existing discharge consents.

OPTIONEERING AND SOLUTION SELECTION

Optioneering was undertaken to identify the most appropriate solution, with two principal options evaluated in detail:

1. Modification of the existing overflow shaft to incorporate mid-level valved inlets
2. The installation of a siphon-based system over the crest of the dam.

Option 1, comprising the retrofit of the existing overflow shaft, was ultimately discounted due to concerns regarding the structural capacity of the existing shaft because of its tall and slender design, in combination with relatively limited steel reinforcement shown on the record drawings.

Creating large openings in the structure and making it more top heavy by the addition of valves and operational platforms was not considered a feasible solution. Also, it was not considered feasible to create the new openings in the existing shaft underwater and so would have required a substantial drawdown for construction, which could not be accommodated due to the reservoir's significance as a strategic water resources asset.

In contrast, the siphon solution had fewer structural concerns, required minimal intrusion into the embankment and allowed the reservoir to operate at its normal levels throughout construction.

The selected solution comprised three large-diameter steel siphon pipes (DN 1700 mm to DN 1600 mm) installed over the dam crest. The pipes were configured to operate independently, providing redundancy and operational flexibility. The siphons were complemented by an uprating of the existing bottom outlet works (a new 'R09' washout valve), enabling improved drawdown capacity beyond the operational range of the siphons.

DESIGN

The hydraulic design of the siphons was a central component of the scheme and required detailed analysis to ensure stable and reliable operation across the full operating range. The most significant operational and design drivers and challenges that influenced the detailed design solution are summarised in the sub-sections below.

Siphon hydraulics

It is generally accepted that siphons installed below reservoir top water level require an isolation valve in the crown section to prevent accidental priming or spills, and to facilitate safe maintenance. Public access along the dam crest and the associated challenges of security and access for maintenance of any such valves meant that a key objective was to set the siphon crown invert above FSL, thereby eliminating the need for any such isolation valves. Setting the siphon crown relatively high creates hydraulic challenges but analyses demonstrated that this could be done while still maintaining a 5m siphon drawdown range and satisfying drawdown performance requirements.

Once the siphon crown level was confirmed, the required downstream hydraulic levels could be confirmed, enabling the selection of a suitable siphon location, where downstream ground levels were favourable for construction of a discharge chamber.

CIRIA guide C813, Siphons in Dams, sets out a number of siphon 'rules'. These were assessed at Bewl as follows:

Bewl Drawdown Improvements Scheme: Design, Delivery and Commissioning of a Large-Scale Siphon System

- ***Inlet submergence:*** Sufficient inlet submergence is required to avoid air entrainment from vortices causing a reduction in capacity, which, if severe enough, may result in column separation 'breaking the siphon' causing flows to cease. Submergence was checked using both the Vischer & Hager Equation and HR Wallingford Equation. Due to uncertainties in the cumulative effects of the three inlets, a CFD analysis confirmed a low risk of vortex creation. An anti-vortex hood was added on top of the inlet cages for additional confidence.
- ***Maximum sub-atmospheric pressure:*** Hydraulic analysis and preparation of a hydraulic profile demonstrated that the maximum sub-atmospheric pressure at the siphon high point was within the suggested limit of -7.9m (gauge) with reservoir water level at the bottom of the siphon range. This gave confidence that the system could operate across its full operating range without the water vaporising and breaking the siphon effect.
- ***Balance of frictional losses:*** Head losses in the downstream leg of the siphon were shown to be greater than the losses in the upstream leg, which is considered favourable as it prevents the siphon downstream leg of 'draining' empty and promotes stable and continuous operation.
- ***Minimum velocity:*** Minimum velocities are required to move bubbles or air pockets forward along downward sloping pipes and stop them from climbing against the flow. Analysis showed that, at Bewl, the minimum velocity was sufficient to avoid air collecting at the highpoint, which may have otherwise reduced capacity or broken the siphon effect.

Siphon Priming Arrangement

The priming equipment at Bewl is contained in a concrete priming chamber set into the downstream side of the dam crest. Using experience from previous large siphon projects, it was decided to adopt a dual-priming system comprising two phases as follows:

- ***Phase 1:*** Water priming of downstream siphon leg using self-priming pumps, drawing water from the reservoir. This is carried out with the submerged discharge valves (SDVs) at the downstream end closed and with the 300 mm diameter air valves in the crown section open.
- ***Phase 2:*** Closure of the air valves and vacuum priming of the crest and upstream leg, drawing water from reservoir.

Operation of the siphons can be carried out from local control panels situated within the MCC Kiosk adjacent to Discharge Chamber or from a

remote kiosk on the left abutment. The installation of the remote kiosk was implemented to ensure siphon operation can be controlled from a safe location during emergency conditions. Following HAZOP and LOPA assessments, it was generally decided to have a duty/standby arrangement for most critical plant.

Siphon pipe configuration and structural design

Due to their large diameter, a decision was made early in the design phase to use welded steel for the siphon pipes. However, as the design developed, flanges were added in a number of locations to aid construction and provide future-proofing against possible changes to reservoir top water level.

It was decided to adopt above-ground siphon pipework (except through the embankment crest) to minimise intrusion into the reservoir embankment, and to improve constructability.

Pipe stresses were assessed using proprietary software for load combinations in steady-state and transient conditions and also for three environmental external pipe loading scenarios as follows:

- Wave slam loading to the marine section of the siphon pipework
- Wind loading
- Hot summer and cold winter thermal effects

Of the three conditions, thermal expansion and contraction proved most onerous and, to mitigate these stresses, pipe saddles with low-friction PTFE slide bearings were used for both the upstream (underwater) and downstream siphon supports.

Following contractor involvement regarding underwater works on the upstream side, it was agreed to install pipe saddles on 'm-shaped' precast units, place on the concrete revetment slabs and to fill precast voids with in-situ concrete to maximise friction and minimise the use of dowels that might compromise the integrity of the revetment. Pipe saddles on the downstream side were installed on sloped concrete pads set into the embankment, with each pad having four screw piles to distribute loads into the dam shoulder and avoid the potential for localised shallow slips. Pipe saddles downstream of the embankment toe, towards the discharge chamber, were founded on horizontal in-situ concrete plinths. With the siphon crest and discharge chamber being two fixed points, a double bellows expansion joint was included at the downstream toe to allow for expansion and contraction and constructability.

Bewl Drawdown Improvements Scheme: Design, Delivery and Commissioning of a Large-Scale Siphon System

Discharge chamber

The River Bewl is a small watercourse capable of passing flows of less than $1\text{m}^3/\text{s}$ in normal conditions. As such, it is necessary to have good control of siphon flows during testing. Submerged discharge valves were included for this purpose (in conjunction with a test-water retention facility, as described later).

The discharge chamber comprises three individual stilling chambers discharging into a common collection channel that feeds the discharge culvert. The stilling chambers were designed in accordance with valve supplier guidance. The discharge chamber is 12m long, 16.7m wide and 10.8m tall with the upper 3m being above finished ground level.

Discharge culvert

The siphon location, in combination with local topography and steep valley sides, resulted in the need for a steep culvert to convey flows from the siphon discharge chamber to the existing spillway chute. To mitigate the potential for high flow velocities, internal energy dissipators were incorporated along the culvert base. The culvert hydraulics were modelled using the open-source software, *HY-8*, and confirmed by CFD analysis during later stages of the detailed design phase.

The culvert is 4m wide and 2.5m high with a full-flow capacity of approximately $46\text{m}^3/\text{s}$, which exceeds the maximum possible combined discharge from the siphons. Due to its size and criticality as a water conveyance structure, it was not considered appropriate to design the culvert using precast units. Instead, a composite design was adopted with in-situ concrete base and sidewalls and pre-cast roof slabs with an in situ structural topping. This design reduced the requirement for temporary works for roof construction, thus providing programme savings.

Figure 1 below shows a section through embankment and siphon pipework.

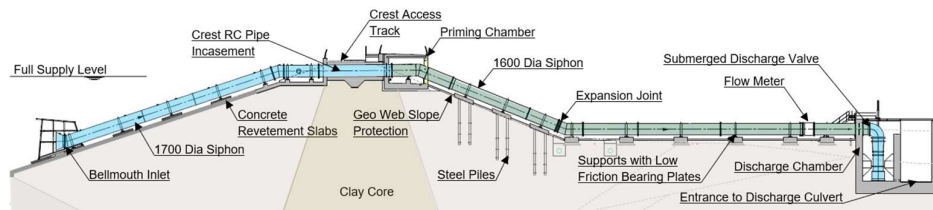


Figure 1: Section through embankment and siphon pipework

Storage and controlled release of siphon test water

The Qualified Civil Engineer for the Bewl Drawdown Scheme stipulated that provisions should be made to regularly test a single siphon at full flow (i.e.

up to 12.5 m³/s) for at least thirty seconds. As noted above, the River Bewl has sensitive habitats that would most likely be affected by flows > 1m³/s.

To address this, the existing spillway chute was converted into a siphon test-water retention basin by constructing a concrete weir across its downstream sill. This weir is fitted with multiple penstock gate openings to provide operational flexibility. The weir height was designed to maximise the test-water storage capacity while also ensuring enough freeboard over the top to pass emergency drawdown releases or the Probable Maximum Flood.

Because the available test-water storage volume was constrained by the dimensions of the existing spillway (and the height of the new weir), it was necessary to minimise the volume of test-water by minimising the opening and closing time of the submerged discharge valves.

Through collaboration with the SDV supplier (Glenfield Invicta), hydraulic actuators were adopted to achieve opening and closing times of 60 to 80 seconds. This, along with the siphon-break air valves in the crown, were modelled using proprietary transient hydraulic modelling software to ensure that these valve movements did not induce excessive surge responses. The output from the transient analyses were incorporated into the pipeline structural analysis.

Environmental Design Considerations

The main environmental considerations during the design stage related to the site's status as Area of Outstanding Natural Beauty and the sensitivity around landscape and visual aspects as well as the local ecology and the need to protect the downstream watercourse from excessive siphon test flows.

The selection of a buried discharge culvert rather than an open channel improved visual amenity and avoided severing the habitat area. The downstream pipes were specified to have olive green painted finish on the outside to reduce visual impact. DWI Reg 31 restrictions limited the external paint options for the upstream (marine) pipes so these were kept as the standard aquamarine colour.

To make the priming facilities as unobtrusive as possible it was decided to fully contain them within a concrete chamber, thus making the design inherently safe and avoiding the need for visually intrusive security fencing on the crest of the dam. The roof of the chamber was therefore designed to be accessible to the public as a viewing platform with ramps up to roof level to make it accessible to all. The downstream face of the priming chamber was formed from louvre panels to reduce the use of concrete and to improve ventilation and avoid the chamber being a confined space.

Bewl Drawdown Improvements Scheme: Design, Delivery and Commissioning of a Large-Scale Siphon System

CONSTRUCTION

Construction, which commenced in 2023, faced several challenges, including restricted access across the dam crest, the need to operate within an environmentally sensitive area and the logistical complexity of installing large components within a constrained site.

The dam crest is a public footpath, which was only closed during the dam crest excavation and reinstatement works. The access road at the dam toe of the dam which forms a section of the *Sussex Border Path* was kept open to the public through the works.

Construction of the discharge chamber and culvert were carried out within open excavations with all cut slopes and stockpiles being covered in plastic sheeting to minimise rainfall erosion that would run-off to the River Bewl, creating a pollution risk. The culvert tie-in to the existing spillway, the retention weir and the R09 washout works were constrained to periods when augmentation flows were not being released to the spillway (June to August).

Construction of the pipes through the crest of the embankment was carried out in autumn/winter 2024 (see Figure 2). To be sure of achieving a fully reinstated dam well ahead of rising reservoir water levels, a design change was instigated where reconstruction of the upstream shoulder was done predominantly with mass concrete to minimise the need for difficult placing and compaction of earthworks materials during the winter months.

Marine works to install the pipes on the upstream slope of the dam were carried out during spring and summer 2025 (see Figure 3).



Figure 2. Installation of pipes through dam, to priming chamber (Oct 2024)



Figure 3. Lifting and floating of marine side pipes (Aug 2025)

Despite these challenges, the works were successfully delivered without significant disruption to reservoir operations or the surrounding environment, and with public access maintained at all times.

Figure 4 shows an aerial view of the constructed siphon works, conveyance system and uprated outlet works. Figure 5 provides a closer view of constructed uprated bottom outlet works (R09).



Figure 4: Aerial view of constructed drawdown scheme



Figure 5: Uprated Outlet works (R09)

TESTING AND COMMISSIONING

Commissioning was undertaken between April and May 2026 and included full-scale testing of all three siphons and the uprated bottom outlet works (R09). Flow measurements and reservoir levels were recorded to validate hydraulic performance.

The results demonstrated that the siphon system consistently exceeded design expectations, with flows approximately 5.4% higher than predicted,

Bewl Drawdown Improvements Scheme: Design, Delivery and Commissioning of a Large-Scale Siphon System

while the uprated outlet works achieved flows approximately 5% above design values. At full supply level, the combined calibrated hydraulic models achieved a total discharge of approximately 43.3m³/s.

Following the calibration of siphon models using recorded commissioning flows, verification checks confirmed that the hydraulic design satisfied all key performance criteria over the full operating range.

Drawdown analysis based on the calibrated models indicates that the system should be capable of lowering the reservoir level by 5m within 4.4 days, thereby satisfying the statutory requirement.

Results of drawdown analyses based on design flow models as well as calibrated models can be viewed in Table 1 and Figure 6 below, showing the drawdown curves of the siphons in combination with the R09 uprating of existing outlet works. Noted that design flows were calculated by assuming loss coefficients for SDV at 93%-95% open and moderate internal pipe lining wear.

Table 1. Drawdown Summary – Design vs Commissioned

Ref	Siphons @FSL	Bottom Outlet @FSL	Total Max Flow	Drawdown Top 5m	Drawdown to 25%
Description / Units	m³/s	m³/s	m³/s	days	days
Existing	-	1.86 ¹	1.86	94	177
Design	37.5	3.45 ²	40.94	4.7	47
Actual	40.68 ³	3.63 ^{2,3}	44.33	4.4	45

¹ RO6 and RO7 washouts

² New RO9 washout

³ Measured with permanent inline magnetic flowmeters during commissioning and calibrated to FSL.

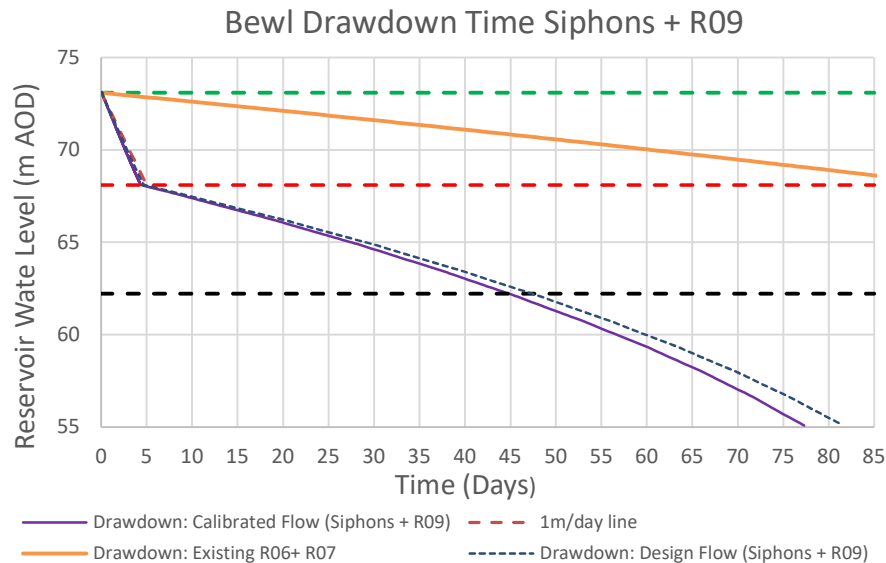


Figure 6: Drawdown Curves

The drawdown rates presented above are consistent with expectations, with actual measured discharges showing a slight improvement relative to the design flows. This provides enhanced system performance and improved drawdown efficiency compared to the operational objectives of the Bewl Drawdown Improvement Project.

It should be noted that the hydraulic models (siphons and R09) were calibrated using test flow data recorded at similar reservoir water levels. It is envisaged that test flows will be recorded in future at varying reservoir levels to provide further validation and refinement of the hydraulic models.

Testing also confirmed that the downstream conveyance and attenuation systems functioned effectively, with no uncontrolled spilling and with adequate freeboard maintained during all test scenarios.

Operational reliability was demonstrated through successful priming, stable operation and effective performance of automated safety systems, including siphon break valves and discharge controls.

KEY LESSONS LEARNED

The Bewl Drawdown Project provides useful lessons for future drawdown improvement schemes. Early engagement with the contractor was critical to address constructability risks associated with underwater working and the planning of dam excavation works around seasonal water levels. The adoption of a dual priming system and setting of siphon crown invert above FSL level enhanced reliability and reduced operational complexity.

Bewl Drawdown Improvements Scheme: Design, Delivery and Commissioning of a Large-Scale Siphon System

Environmental constraints, while challenging, drove innovation in the design of testing and discharge arrangements. Finally, the importance of integrating testing requirements into the initial design proved to be a valuable decision.

CONCLUSIONS

The Bewl Drawdown Improvements Scheme has delivered a robust high-capacity emergency drawdown system that marginally exceeds design requirements. The new facilities achieve the required drawdown of 5 m within five days, providing a significant improvement in emergency response capability.

Importantly, the siphons can be tested without adverse impacts on the downstream infrastructure or environment. The project demonstrates the effectiveness of siphon-based solutions for enhancing drawdown performance in large embankment reservoirs and provides a valuable reference for similar schemes in the future.

REFERENCES

- Defra/EA (2017). *Guide to drawdown capacity for reservoir safety and emergency planning*, Environment Agency, Bristol, UK
- CIRIA (2023). *Siphons in Dams (C813)*. CIRIA, London, UK
- HR Wallingford (2005). *Air Problems in Pipelines*. H R Wallingford, Oxon, UK
- Vischer D and Hager, W (1998). *Dam Hydraulics*. Wiley, New Jersey, USA
- HY-8 (version 7.8 July 2019) produced by Pennsylvania State University in cooperation with FHWA (Federal Highway Administration, USA)



Weir discharge coefficients for reservoir flood studies: values, methods, and effect on reservoir flood levels

MIGUEL M PIEDRA, Mott MacDonald

JENNIFER HOLLOWAY, Mott MacDonald

ANETA CZECH, Mott MacDonald

SYNOPSIS

The discharge capacity of a dam spillway weir is a key component for the assessment of safety conditions of reservoirs during floods. It is common to apply a simple weir discharge equation, using a constant discharge coefficient C to generate the corresponding discharge curve, since site-specific approaches, such as Computational Fluid Dynamics or physical modelling, may not be economically justified. The values selected for C tend to be taken from textbooks for standard weir types or based on experience. There is a wide variety of shape, size and arrangements of spillway weirs in dams, and the selected C may not either be entirely applicable to the subject structure or be better represented using variable C . Often these C values cannot be validated, which introduces a degree of uncertainty in the estimated reservoir levels. Increasing confidence in these calculations is beneficial due to the potential costly reservoir upgrades required resulting from these studies.

This paper presents a range of sources and methods to generate values of C for typical dam weirs based on published documentation, and it explores the effect on the estimated flood rise obtained with two different C values for a real structure, which are arguably equally valid, resulting in considerably different outcomes for the client.

GLOSSARY

A range of different resources informed this paper; with variations in the symbols used for equation inputs. For simplicity, symbols have been standardised as below:

- C Weir coefficient, when used in $Q = C b h^{1.5}$ (e.g. equation 3), generally $1.5 < C < 2.2$.
- C_d Discharge coefficient, when used in $Q = \frac{3}{2} C_d b \sqrt{2g} H^{1.5}$ (e.g. equation 2a), value < 1.0 , generally ~ 0.6 .
- g length of the weir crest (in the direction of the flow), see CIRIA TN134.

Weir discharge coefficients for reservoir flood studies: values, methods, and effect on reservoir flood levels

- h Flow depth (m), measured some distance upstream of the weir.
- H Total head (m), including the velocity energy component.
- Q Discharge (m^3/s).

INTRODUCTION

The discharge capacity of a dam spillway weir is a key component for the assessment of safety conditions of reservoirs during floods. It is rare that the discharge capacity curve of the structure is available, so new flood assessments must develop a discharge curve for the spillway weir.

It is common to apply a simple weir discharge equation, using a constant discharge coefficient C^1 , since site-specific approaches, such as computational fluid dynamics or physical modelling, may not be economically justified. In general, the values of C increase with head and decrease with the height of the weir (Horton, 1907); so, a constant value may underestimate flows at higher depths. The values selected for C tend to be taken from standards or textbooks, or based on experience, without the ability to validate the choice made. This introduces a degree of uncertainty in the estimated weir hydraulic capacity and subsequent peak reservoir levels.

Since some flood studies may lead to costly upgrade works, increasing confidence in the estimation of spillway discharge capacity is seen as a benefit for our industry and good management of reservoir managers' funds.

This paper presents a summary of the equations used for calculating weir discharge capacity, key parameters and their values, and discusses a case study in which two values of the discharge coefficient for the same weir, both arguably acceptable, result in very different assessment of reservoir flood conditions and the need for spillway upgrades.

¹ Note that there is a distinction between what we are calling in this paper 'weir coefficient' C , applied to $Q = C b h^{3/2}$, for which $C > 1$ (commonly 1.5-2.2), and the 'discharge coefficient' C_d , when applied to equations 1-2, when $C_d < 1$ (generally ~ 0.6 , as used by Hamill (2001) and Ackers *et al.* (1978)). However, CIRIA TN134 (Ellis, 1989) calls discharge coefficient C_d when applied to $Q = C_d b h^{3/2}$. So, clarity is needed when applying these terms. To avoid confusion, as outlined in the glossary, in this paper we use C when referring to the equation $Q = C b h^{3/2}$. Regardless the name or symbol used, the coefficient is a factor modifying $b h^{3/2}$, to return the calculated discharge of the subject weir. C (or C_d) therefore represents the hydraulic 'efficiency' of the structure.

TYPES OF WEIRS COMMON IN SPILLWAYS

There is a wide variety of weir geometries in spillways encountered in UK reservoirs. They include for instance, 'no weir' (the flowing threshold is integrated within a slab on the spillway bay, marked simply by a high line across the channel), short vertical walls, cropped crump weirs and ogee weirs. The weir equations however (see next section), relate mostly to three types of weir cross sections:

- Broad-crested weirs
- Ogee weirs
- Thin-plate weirs

A broad-crested weir (Figure 1) is a flat-crested structure with a crest length (L) that is large compared to the flow depth (h) ($L/h > \sim 3$, Hamill, 2001). When the crest is 'broad' enough for the flow streamlines to be parallel to the crest, the pressure distribution above the crest is hydrostatic and the critical flow depth is observed on the weir crest (Chanson, 2004). CIRIA TN134 (Ellis, 1989) indicates that a weir coefficient $C \sim 1.7$ applies to this type of weir. However, note that if the upstream edge of the weir is rounded, the value of C increases, compared to the same weir with sharp upstream angle (see Chapter 7 in CIRIA TN134).

Ogee weirs, also in different configurations, have a profile that closely resembles that of the lower nappe of the flow passing over a sharp-crested weir, so the flow tends to 'glide' over the weir profile, with increased efficiency. Thus, the values of C associated with this weir are higher (> 1.85 , see Figure 3.7 in CIRIA TN134, usually taken as 2.1-2.2).

Thin-plate weirs are commonly used in measuring devices for small flows (e.g. v-notch weirs). But, if a real-case weir is too short to be considered a broad-crested weir ($L/h < \sim 3$), for instance a weir made of a vertical wall, can a sharp-crested weir equation be applied to it? An analysis made by one of the authors as part of a reservoir flood study found that the discharge curves for a 0.4m thick vertical wall weir using equations for broad-crested and thin-plate weirs (Ackers *et al.*, 1978) converge in similar values over the range of applicability of the equations.

Weir discharge coefficients for reservoir flood studies: values, methods, and effect on reservoir flood levels

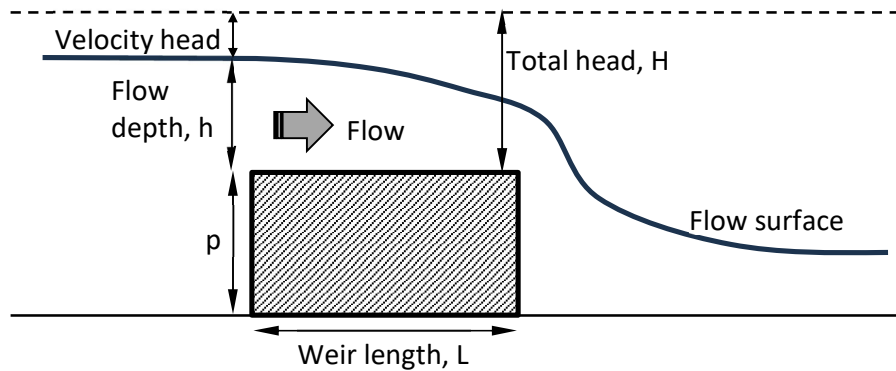


Figure 1. Main parameters of weir geometry and flow.

The height of the weir (p) also influences discharge capacity. The larger p is in relation to the flow depth, the less efficient the weir is. Following this logic, low weirs are more efficient than tall weirs, but they also become more easily drowned, which diminishes their hydraulic capacity.

When assessing the discharge capacity of a spillway weir, the above factors must be considered. Without results from physical or detailed computational modelling, the assessor faces a limited choice of approaches. A constant value of C may suffice for an initial calculation, but a more detailed representation is likely to be needed to carry out flood routing calculations that underpin the assessment of freeboard and flood safety conditions at reservoirs. The example described in this paper shows a typical case where a spillway weir did not fit any of the cases presented in literature and a choice of value was made. A later review of the calculations used the geometry components as inputs to a 1D computer model to generate a discharge curve for the weir, with different results.

WEIR FLOW EQUATIONS AND DISCHARGE COEFFICIENT

The assessment of the discharge capacity of overflow structures such as spillways generally requires the application of the equation of free flow over a weir.

For a thin-plate, rectangular-profile weir, Ackers *et al.* (1978) proposed:

$$Q = \left(\frac{2}{3}\right)^{3/2} C_d b \sqrt{g} H^{3/2} \quad \text{equation 1a}$$

With Q being the total discharge over the weir, C_d is the coefficient of discharge (Ackers *et al.* used C_p , we have replaced it with the more common

symbol Cd), b is the effective breadth of the weir (also called weir length²), g is the gravitational acceleration, and H is the total head relative to crest level (Figure 1). Since H is not possible to be ascertained in advance, as it includes the velocity energy term, an alternative equation, using h (gauged depth) is presented by Ackers *et al.* (1978):

$$Q = \left(\frac{2}{3}\right)^{3/2} Cd Cv b \sqrt{g} h^{3/2} \quad \text{equation 1b}$$

With Cv being a dimensionless coefficient to account for the effect of the approach velocity (see more details in Ackers *et al.* (1978) and Novak *et al.* (1996)).

For a broad-crested weir, flow capacity is defined using equation 2 (Hammil, 2001; Chadwick and Morfett, 1996 and Novak *et al.*, 1998):

$$Q = \frac{2}{3} Cd b \sqrt{2g} H^{3/2} \quad \text{equation 2a}$$

Which can also be expressed as:

$$Q = \frac{2}{3} Cd Cv b \sqrt{g} h^{3/2} \quad \text{equation 2b}$$

Equations 1b and 2b are commonly simplified as:

$$Q = C b h^{3/2} \quad \text{equation 3}$$

With C being the ‘weir coefficient’, which is what is generally selected by the user.

For simple calculations, a constant value of C is commonly adopted. Several sources offer values for a range of weir geometries (see next section), generally ranging from 1.7 associated with broad-crested weirs to 2.2 for ogee profiles. However, the value of C varies with flow depth (Ellis, 1989; Addison, 1944; Novak *et al.*, 1998), increasing its value with increasing depth of flow, and decreasing with the height of the weir.

In practice, the definition of C , either constant or variable, is rarely supported by either physical or computer modelling, so the values adopted are not validated.

² In this paper we are adopting ‘breadth’ or ‘width’ of the weir, as its alignment is across the direction of the flow, to avoid confusion with ‘length’, which is taken as the dimension of the weir in the direction of the flow. For how to calculate the effective width, see for instance chapter 4 in Novak *et al.* (1996) and chapter 13 in Chadwick and Morfett (1998).

Weir discharge coefficients for reservoir flood studies: values, methods, and effect on reservoir flood levels

SOURCES OF WEIR COEFFICIENT, C

Below is a selection of sources of C :

- a) Part B of CIRIA Technical Note 134 (Ellis, 1989) includes plots of the values of C (when using $Q = C b h^{3/2}$) obtained from laboratory experimentation for a range of weir geometries (cross sections) and the ratio of p/h , with p being the height of the weir crest at the upstream face and h the depth of flow, measured some distance upstream of the weir. The values of C are generally either constant or decrease with increasing p/h , meaning that the efficiency of the weir reduces for low depths.
- b) Ackers *et al.* (1978) also offers plots and equations to calculate the value of C (C_p in the original publication) based on previous researchers. These plots use h/L and $h/(h+p)$ (L is the length of the weir in the direction of the flow) to return a correction factor F to apply to $C = 0.848F$. The ranges for these variables relate to laboratory experimentation and may not cover the real subject structure being analysed. Therefore, the user is presented with the need to inter/extrapolate values.
- c) Novak *et al.* (1996) contains a table (Table 8.1) summarising the equations and coefficients applicable to a small range of weir geometries, in which C_v depends on C_d . For the values of C_v , they refer to, amongst other authors, Ackers *et al.* (1978).
- d) BS 3680-4F- *Round nose horizontal broad-crested weirs* deals with the measurement of flow in rivers and artificial channels under steady flow conditions using round-nose horizontal broad-crested weirs. This standard may be used to assist calculations for this type of structure, although access to it may be limited by its high cost.
- e) Chadwick and Morfett (1998) also offer values for C_d (when applying equation 1b, including an approximation to F ($C_d = 0.848F$, as per Ackers *et al.*, 1978) for a limited range of h/L and $h/(h+p)$. The ranges of weir size/geometry are small and may not cover the real-case weir analysed. Once again, if the study weir characteristics are beyond the ranges, the user is faced with the decision of extending the calculations beyond these ranges.
- f) Horton (1907) contains the results of the experiments by multiple authors with a wide range of weir geometries (cross sections), and it

presents equations applied to thin-plate and broad-crested weirs. The values of C correspond to American units, but their values may be a valuable source if transformed to use with SI units.

Based on the above, it appears that Ackers *et al.* (1978) is referenced by several of the above sources, resulting in Ackers *et al.*'s work becoming a primary source.

CASE STUDY

Background

In 2019, an S47 inspection was undertaken at a Category B Scottish reservoir under the Reservoirs (Scotland) Act 2011. This resulted in a measure in the interest of safety (MIOS) relating to the condition of the auxiliary spillway, potentially requiring refurbishment works. Various studies and investigations were undertaken, resulting in a recommendation to construct a new auxiliary spillway.

Much of the evidence for this recommendation was sourced from a flood study completed over 10 years prior. Therefore, Mott MacDonald (the Consultant) was commissioned to undertake a new flood study to assess the adequacy of the two existing spillways (main and auxiliary) to pass a 1-in-10,000-year flood event.

The main spillway consists of a concrete weir with an unusual triangular profile with a vertical downstream face, represented in Figure 2, overflowing into a rough channel cut into rock. The auxiliary spillway is set at a higher elevation than the main weir, and it comprises a wide, flat concrete sill, followed by a stepped cascade built over the main embankment, overflowing into a narrow channel below. Both spillway channels are spanned by footbridges supported by piers at regular intervals across the overflows.

Weir discharge coefficients for reservoir flood studies: values, methods, and effect on reservoir flood levels

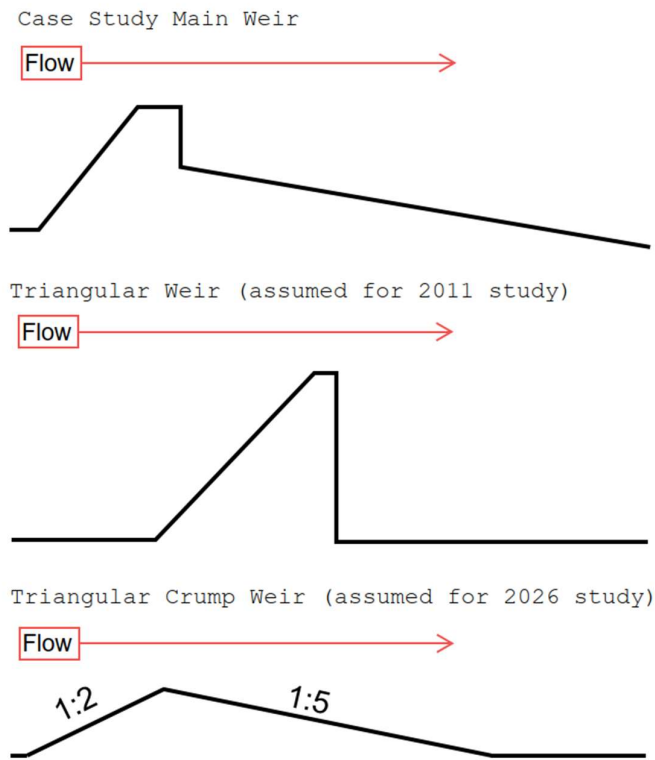


Figure 2. Sketches of main weir (top); closest case in CIRIA TN134 assumed to have been used by the 2011 flood study (middle); and crump weir case adopted by the initial calculations for the 2026 flood study (bottom).

2011 Flood Study

Auxiliary Weir

Due to the complex geometry of the auxiliary spillway, a physical model was constructed in 2011. This model provided a C value of 1.43, which was then applied to produce the stage discharge relationship used during the flood study.

This value is similar to the standard value of 1.44 resulting from the rectangular profile weir equation, Scenario 6 Figure 7.10(a) provided in CIRIA's Design of Flood Storage Reservoirs (1993) guidance.

$$Q = 0.46b\sqrt{gH^{3/2}} \quad \text{Equation 4a}$$

Main Weir

The 2011 flood study applied a constant C value of 1.98 for the main spillway. The flood study report states that this is supported by Figure 3 in CIRIA

Technical Note 134, but the specific case was not stated – highlighting the need for detailed and accurate flood study reporting. This is the same C value that was applied during a previous flood study in 1999. There are no further discussions regarding the selection of the coefficients within the 2011 flood study report.

2026 Flood Study

The Consultant began a new flood study in 2025. The C values applied by the 2011 flood study were used as a starting point for the assessment.

Auxiliary Weir

The previous physical modelling of the auxiliary weir provided a well-founded estimate of the discharge coefficient of this existing structure. However, the new auxiliary weir being proposed has an ogee weir profile; a C value of 2.10 was adopted. This value was selected in accordance with Scenario 9 Figure 7.10(b) CIRIA (B14) Design of Flood Storage Reservoirs (1993) using the following equation:

$$Q = 0.67b\sqrt{g}H^{3/2} \quad \text{Equation 4b}$$

Main Weir

The selection of a weir coefficient for the existing main weir was a more complex process due to the weir profile. In this flood study, the triangular profile crump weir equation included in Figure 7.10(b) CIRIA (B14) Design of Flood Storage Reservoirs (1993) was applied (Figure 2).

$$Q = 0.633b\sqrt{g}H^{3/2} \quad \text{Equation 4c}$$

This resulted in a C value of 1.98; in line with the 2011 flood study (thought to correspond to case 5 in CIRIA Technical Note 134) and the 1999 flood study, for which the method for obtaining C is unknown.

The crump weir most closely resembled the main weir profile (Figure 2) out of the reservoir control structures detailed in CIRIA (B14) Design of Flood Storage Reservoirs (1993). Due to the complex geometry of the main weir, calculations of weir angles and p/g (weir height/crest width) did not align with any of the cases detailed in CIRIA Technical Note 134; therefore, these were thought to not be representative.

2026 Flood Study Check

The outcome of the 2026 flood study was that the proposed new auxiliary spillway was unnecessary, on the basis that the downstream channel be formalised and the freeboard be enhanced to provide future settlement allowance. The Client requested confirmation of this conclusion, as it contradicted their previous assessments. This led to an independent,

Weir discharge coefficients for reservoir flood studies: values, methods, and effect on reservoir flood levels

internal review of the calculations by the Consultant. A separate team carried out a fresh assessment of the 1-in-10,000-year flood event hydrographs and flood routing, ignoring the previous work, assuming only the existing main spillway was operational.

To eliminate user bias from the selection of C resulting from the choice of the 'most similar' case in CIRIA TN134, the main spillway weir was represented in Flood Modeller using a weir unit. The crest elevation, width, and the height of the weir crest above the bed at the upstream and downstream ends (p_1 and p_2) were inputs, although the upstream and downstream slopes of the weir are not accounted for by Flood Modeller. It was anticipated that the model would use the geometry values to generate a specific discharge curve. The curve generated from this representation was found to correspond to an equivalent weir coefficient of $C=1.7$, characteristic of a broad-crested weir. The resulting peak reservoir level obtained using this approach was high enough for the auxiliary spillway to be required. This was in contradiction with the previous assessment.

Upon further review, a weir coefficient of 1.7 was considered too low, as the weir geometry does not conform to the classical definition of a broad-crested weir, and its low height favours a higher C (see section TYPES OF WEIRS COMMON IN SPILLWAYS). After internal discussions, the value of $C=1.98$ from the earlier assessment was adopted.

The analysis highlights the significant influence of the discharge coefficient on the resulting outputs. In the absence of specific values applicable to the subject weir geometry, its selection is primarily reliant on the engineering judgement of the analyst.

DISCUSSION

The 2026 flood study concluded that an auxiliary spillway is not required to pass a 1-in-10,000-year flood; assuming that the downstream channel of the main spillway is formalised, and the freeboard be enhanced to allow future settlement allowance. The decision on the need for the auxiliary spillway is critical to the safety of this reservoir. To reduce uncertainty over the main weir rating curve, the Consultant recommended to the Client undertaking physical scale modelling of the main spillway. This work is under procurement at the time of writing this paper.

The case study presented shows that the selection of the discharge coefficient for use in a weir flow equation can become critical to the results obtained. It is uncommon for a discharge curve to be based on a specific analysis to be available, so the adoption or estimation of values of C is often

required. Some references provide a single value of C for the type and geometry of the structure. Others provide the means of calculating a variable C , but the range of applicability can be insufficient for the subject structure being ascertained. CIRIA TN134 guide is very useful and covers some unusual weir geometries, but the cases presented are limited. If the subject weir is different to those, the user can either discard the plots or attempt to 'estimate' the value of C by inter/extrapolating the values available using the most similar case. The options in CIRIA (B14) guide can also be explored. As outlined earlier, the ratios h/p , h/L , p/g and the roundness of the weir upstream edge all affect the efficiency of the weir, so assimilating the subject weir to one of the cases in CIRIA TN134 may be inappropriate.

For the case study presented, two different weir coefficients were applied to the same structure, originating from different approaches and teams undertaking the work. This demonstrates the uncertainty over which discharge curves to adopt when a site-specific one is not available. The decision on the parameters to adopt is made more difficult when the subject weir does not align with the cases presented in available references. A decision must be made whether to proceed with the values taken from references (unconfirmed), or to undertake further investigations, providing a greater level of certainty of the discharge capacity of the spillweir. This decision should be informed by:

- the risk category of the dam;
- the location of the weir, e.g. whether it is "on dam" or "off dam", informing the risk of embankment failure;
- the sensitivity of the design calculations to the discharge coefficient selected; and
- the extent/cost required to undertake further investigations.

CONCLUSION

The case study presented provides a clear example of how sensitive flood studies can be to the choice of weir coefficient, particularly where no published values exist for the specific weir geometry. The study shows that even relatively small changes in the adopted coefficient can lead to significant differences in predicted performance, with direct implications for design decisions and outcomes.

Given this level of sensitivity and the associated uncertainty, a physical model is to be constructed for the main weir at the site presented. This will help improve confidence in the assessment, not only for this reservoir but potentially also for other sites with similar weir geometries, which are not

Weir discharge coefficients for reservoir flood studies: values, methods, and effect on reservoir flood levels

well represented in existing literature. Some sources of weir discharge coefficients are provided in this paper.

Sufficient attention and time should be dedicated to ascertaining a suitable discharge curve for the spillway weirs being analysed, which, as demonstrated, can have a major impact on the outcomes of flood studies and the need to undertake costly upgrade works.

REFERENCES

- Addison H (1944). *A treatise on applied hydraulics, 3rd edition*. Chapman and Hall.
- Ackers P, White W R, Perkins J A and Harrison A J (1978). *Weirs and flumes for flow measurement*. John Wiley & Sons.
- Chadwick A and Morfett J (1998). *Hydraulics in civil and environmental engineering, 3rd edition*. Spon Press.
- Chanson H (ed.) (2004). *Hydraulics of Open Channel Flow (Second Edition)*. Butterworth-Heinemann.
- Chow V T (1959). *Open-channel hydraulics*. McGraw Hill, reprint by The Blackburn Press.
- Ellis J (1989). *Guide to analysis of open-channel spillway flow. CIRIA Technical Note 134*. CIRIA, London, UK
- Hall M J, Hockin D L and Ellis J B (1993). *Design of Flood Storage Reservoirs. CIRIA (B14)*. CIRIA, London, UK
- Hamill L (2001). *Understanding hydraulics, 2nd edition*. Palgrave.
- Horton R E (1907). *Weir experiments, coefficients and formulas. Water supply and irrigation paper no. 200*. United States Geological Survey, Washington DC, USA. <https://pubs.usgs.gov/wsp/0200/report.pdf>
- Novak P, Moffat A I B, Nalluri C and Narayanan R (1996) *Hydraulic structures, 2nd edition*. E & FN Spon.



The British Dam Society

Optimising reservoir flood safety and storage capacity: Comparative design of labyrinth and ogee weirs

PHOEBE ERSKINE, Mott MacDonald
MIGUEL PIEDRA, Mott MacDonald
ALASDAIR WALKER, Mott MacDonald

SYNOPSIS

Loch Liath is a Category A impounding reservoir, which makes up part of the Livishie Cascade hydropower scheme, operated by SSE Renewables (SSE). The dam consists of a rockfill embankment supporting a concrete core wall. The overflow comprises a single, tilting gravity gate, which operates automatically with rising water levels. Notably, the gate cannot be manually operated if it fails to operate as designed.

SE had identified concerns with a flood gate at this location that could not be tested. In addition, the latest Section 47 inspection report, dated March 2022, identified that the gate failing to operate correctly in a major flood is the greatest residual risk associated with the spillway. A flood study, carried out by SSE, indicated that the dam would overflow during a Probable Maximum Flood (PMF) if the gate failed to operate. SSE initially evaluated several fixed crest options for gate replacement, selecting a horizontal crest concrete ogee weir as the preferred alternative, with a lowered top water level. SSE Hydro commissioned Mott MacDonald to develop a detailed design for the preferred solution and to consider further optimisations to limit the loss of storage capacity. A labyrinth weir was analysed, comparing its performance and the resulting weir crest elevation to the ogee weir. The former provides increased discharge capacity, which allows for a higher crest and thus, a reduction of the storage volume lost when lowering top water level.

This paper summarises the assessment history and optioneering for the tilting weir replacement. It also outlines the design considerations for the proposed replacement options, including hydraulic performance, reservoir capacity impacts, and construction and access requirements.

INTRODUCTION

Liath Reservoir is one of the four reservoirs that form the Livishie Cascade, a hydropower scheme built in the 1960s near Invermoriston, west of Loch Ness, Scotland. It is owned and operated by SSE Renewables (SSE), who is the Reservoir Manager. Loch Liath is a natural loch that was raised by an

Optimising reservoir flood safety and storage capacity: Comparative design of labyrinth and ogee weirs

approximately 10m tall concrete-core, rockfill embankment dam. The stored volume is small (1.62Mm^3). The overflow is controlled by a 10.2m wide, single leaf tilting gate, which is self-regulated (*Figure 1*). The top edge of the raised (closed) gate defines the top water level of the reservoir (457.2mOD).

The gate is linked to concrete blocks that rise as the reservoir water fills the float chambers with rising water levels, the gate is balanced so the weight of the reservoir water on its upstream face is necessary to open the gate. This makes testing the operation of the gate a notable challenge, as reservoir levels are highly controlled and there are few opportunities for raising the water sufficiently for the gate to open significantly. The development of a satisfactory process for raising water levels in the reservoir and applying a 'pull' to the gate proved elusive. A moderate reservoir rise in January 2022 showed that the gate did operate, but the measurements of the gate movements provided by the instrumentation did not match the expected travel.

All this posed a challenge for the reliance of the gate to regulate reservoir levels during large floods, and comply with the recommendations by inspecting engineers on gate testing. Further, the site is above 450mOD and liable to ice and snow falls in winter, limiting access to the dam and increasing the chance of freezing of the self-regulating system. Therefore, the correct operation of the gate during the design and safety-check floods could not be guaranteed. The Reservoir Manager had identified these issues and considered options for removal of the gate.



Figure 1. View of Liath dam and gate from downstream

INITIAL STUDY AND ASSESSMENT

Flood study

In 2021 the Reservoir Manager completed a reservoir flood study for the Livishie Cascade. The assessment assumed that the gate at Liath Reservoir would operate following the designed stage-opening rules. Under these circumstances, the dam would have sufficient freeboard during the design event and have a moderate wave overtopping rate during the PMF, the safety check event.

However, the latest Section 47 inspection report (March 2022), identified that the gate failing to operate correctly in a major flood is the greatest residual risk associated with the spillway. A subsequent assessment assuming the gate did not operate found that the dam would overflow during the PMF.

Initial optioneering

To overcome the uncertainty over the gate operation and reduce the risk to the reservoir overflowing, the Reservoir Manager carried out an initial optioneering assessment of lowering the TWL to provide sufficient freeboard. Two options were analysed in detail (*Figure 2*):

- a) partial lowering of the tilting gate and fixing it in place (no longer moving);
- b) replacement of the movable gate with a fixed-crest concrete ogee weir.

Both options would result in a lower TWL. A major advantage of these options is that neither of them requires testing or the maintenance regime that the existing movable gate does.

The performance of the options was analysed using one-dimensional hydraulic computer modelling (Flood Modeller) and the hydrological inputs developed during the 2021 reservoir flood study. The work carried out by the Reservoir Manager, set the new crest level for the options at 455.9mOD (lowering existing gate) and 455.7mOD (replacing gate with ogee weir), resulting in lowering the top water level by 1.3-1.5m, with the subsequent loss of storage and generation potential.

The Reservoir Manager selected the ogee weir as the preferred option.

Optimising reservoir flood safety and storage capacity: Comparative design of labyrinth and ogee weirs

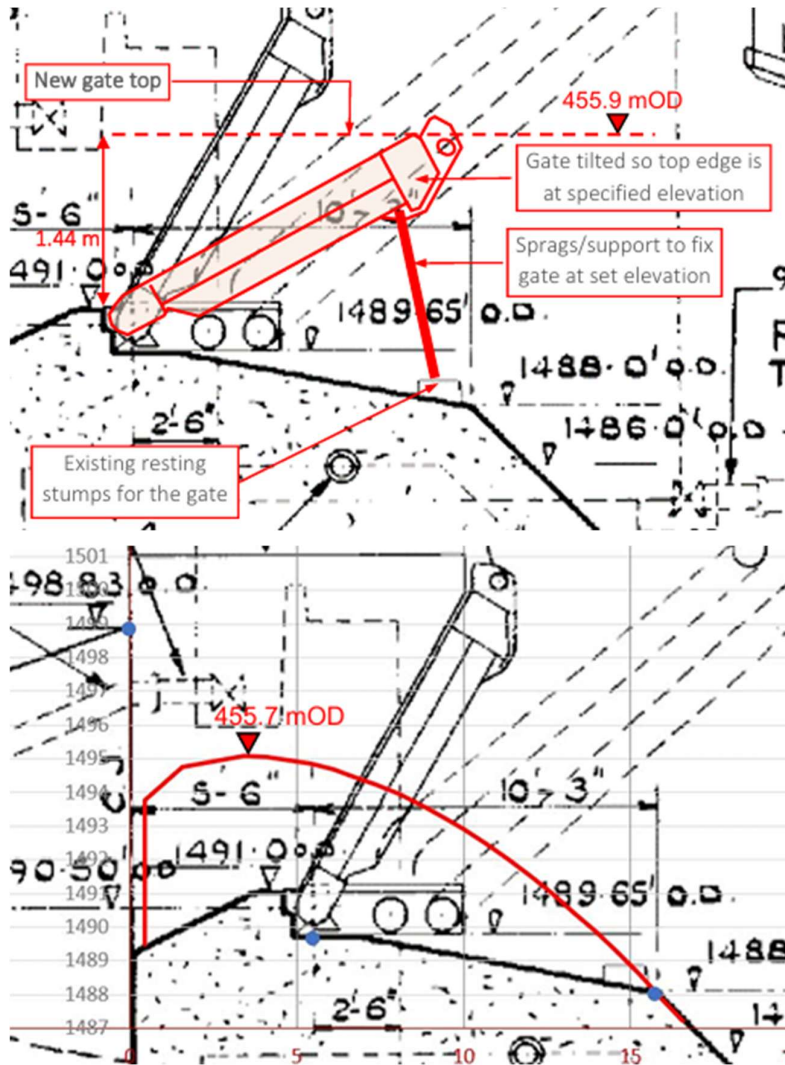


Figure 2. Sketches of the possible options tested by the Reservoir Manager. Top: lowering the tilting gate; bottom: new ogee weir.

HYDRAULIC DESIGN OF PREFERRED OPTIONS

Identification of storage optimisation options

After carrying out their own options study, the Reservoir Manager commissioned Mott MacDonald (Designer) to carry out detailed design of the preferred option of the ogee weir. Additionally, during the proposal stage, the Designer identified an opportunity to construct a labyrinth weir to increase the design top water level and reduce loss of storage capacity, while maintaining reservoir safety in limiting the flood rise to limit wave overtopping. The proposed labyrinth weir option was accepted by the

Reservoir Manager and both options were progressed through to detailed design.

Wave overtopping and flood stillwater level

To set the top water level of the new weir, the wave overtopping carried out by the Reservoir Manager was reviewed by the Designer. In agreement with the Construction Engineer for the design, a tolerable limit of 25l/s/m of wave overtopping was set. This was based on the recommendation in Floods and Reservoir Safety 4th Edition for rockfill dams being able to withstand up to 50l/s/m of wave overtopping. The lower limit was set to include some redundancy in the design to allow for sensitivity of the hydrology and hydraulics, as well as impacts of climate change on the reservoir inflows. The allowable peak stillwater level was set based on this wave overtopping rate and the weir crest level for both options was found by iteration that resulted in this stillwater level.

Ogee weir design

The design of the ogee weir was progressed as a preferred option from the initial flood study carried out by the Reservoir Manager. The weir was represented in the 1D hydraulic model as having a constant weir coefficient of 2.1. The weir crest level was determined by iterating the model with different levels to determine which level resulted in the chosen peak stillwater level for the PMF event. The final weir level for the ogee weir was set to 455.7mAOD.

Labyrinth weir design

Labyrinth weirs have the benefit of having a significant weir length for the width of channel they are constructed within (*Figure 3*). The coefficient of discharge for labyrinth weirs is lower than more commonly found weirs, such as broad crested or ogee weirs, but their capacity is much increased for the same channel width, due to the increased effective weir length that can be achieved.

The rating curve of a labyrinth weir is dependent on the total centreline length of the weir, and the coefficient of discharge. The coefficient of discharge is variable depending on the sidewall angle, and the ratio of the head over the weir to the height of the weir structure (H_T/P).

Optimising reservoir flood safety and storage capacity: Comparative design of labyrinth and ogee weirs

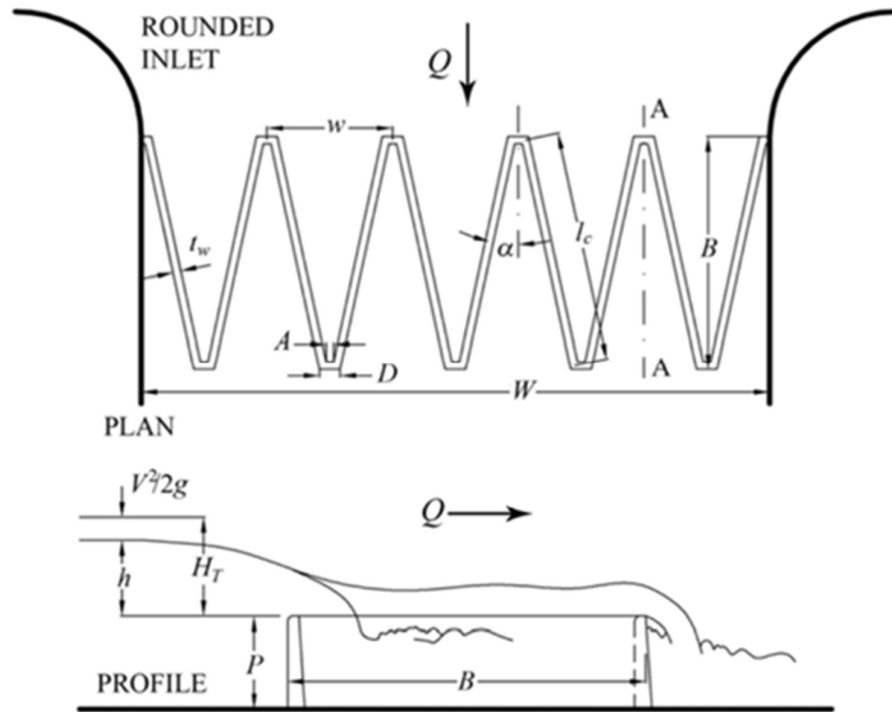


Figure 3. Definition sketch for labyrinth weir geometry, reproduced from ICOLD B172 (2016) and amended for clarity

Labyrinth weir optioneering

The initial phase of designing the labyrinth weir option involved identifying a range of layouts for the labyrinth weir and determining the rating curve for the weirs, based on an assumed stillwater level. Initial rating curves were developed in an Excel calculation spreadsheet using the method outlined in ICOLD Bulletin 172 based on an assumed stillwater level and crest level. This method adopts a fixed coefficient of discharge, based on the PMF stillwater level, which is the lowest coefficient of discharge value that would occur, resulting in a more conservative stage discharge curve.

Four options were tested and were based on fitting the weir into the existing structure, which has an opening size of approximately 10m. The options assumed that the existing concrete base for the tilting gate would be levelled off at a chosen level and the new weir would fit onto this surface. This level was generally chosen based on the resulting stillwater level and the height of weir adopted. Additionally, the weir wall thickness was chosen to ensure the structure would not be too slender and to allow for sufficient reinforcement.

The preferred option for the labyrinth weir was one with two cycles, a height of weir of 3.5m and a sidewall angle of 18°.

Design of preferred option

In the initial optioneering for the labyrinth weir, a constant discharge coefficient was applied based on the peak stillwater flood level during a PMF. This is based on the final ratio of H_r/P . This simplified method was used to determine the most efficient labyrinth layout based on four options and to give an approximate weir level to check feasibility of the option. For the detailed design, a variable discharge coefficient was calculated for varying head over the weir. The equations and design process set out by Crookston & Tullis (2013a,b) were used to derive the stage discharge relationship based on the final geometry.

As with the ogee weir, the top water level of the weir was optimised to result in the chosen design stillwater level, to limit the wave overtopping rate to the agreed acceptable limit. The final weir level of the labyrinth weir option was 456.65m AOD, almost 1m higher than the ogee weir option.

COMPARISON OF DESIGN OPTIONS

Hydraulic capacity

Figure 4 shows the final rating curves used for the design of the ogee and labyrinth weir options. The labyrinth weir passes the same flows as the ogee weir at substantially lower depths over the weir. This allows the top water level to be set at a much higher level than the ogee weir crest with equivalent peak stillwater levels.

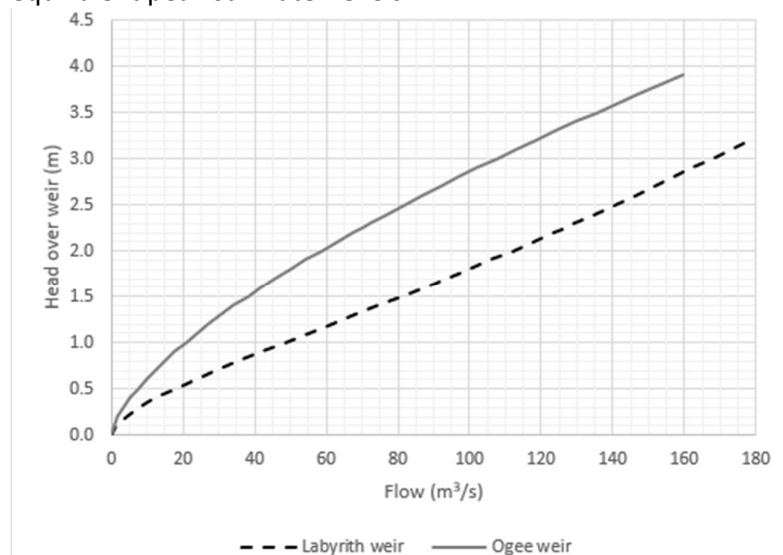


Figure 4. Comparison of stage discharge relationships for ogee and labyrinth weirs.

Optimising reservoir flood safety and storage capacity: Comparative design of labyrinth and ogee weirs

Reservoir storage capacity

Based on the storage curve provided by the Reservoir Manager, estimates were interpolated for storage at the various modelled top water levels. Table 1 summarises the storage capacity of the reservoir for the two options.

Table 1. Summary of reservoir storage capacity for designed options

Option	TWL (mAOD)	Reduction in TWL (m)	Storage (Mm ³)	Reduction from current (Mm ³)	Percentage loss of storage (%)
Current	457.21	-	1.65	-	-
Labyrinth	456.6	0.56	1.48	0.16	10%
Ogee	455.70	1.51	1.20	0.44	27%

The ogee weir option results in a lowering of the current top water level by 1.51m and a loss of approximately 27% of the current storage volume of the reservoir. By comparison, the labyrinth weir results in a much smaller reduction in top water level, only 0.56m, and around 10% loss of storage capacity.

DETAILED DESIGN OF FINAL OPTIONS

The Reservoir Manager requested that a detailed design be undertaken for the ogee weir option and the labyrinth weir option, so that both options could be priced, and a more holistic decision made.

Access constraints

Liath Reservoir is located approximately 4 miles off the A887, and while sections of the access track have been used for wind farm construction, other sections are not suitable for heavy duty construction traffic. This limited the options to rely heavily on pre-cast elements due to both the constraints in getting the units to the reservoir and the constraints in getting a sufficiently sized crane to the reservoir to lift the units.

Due to the height and thickness of the labyrinth weir, 3.5m and 0.5m respectively, it was decided that the pre-cast unit length which could feasibly be delivered to the reservoir and lifted into position would result in too many joints to make the solution practical. The labyrinth weir was therefore, progressed as a cast in-situ option.

The same issues existed for the ogee option. While the weir height was less, the thickness was considerably larger, limiting the linear length for each unit

considerably. The benefits of retaining a pre-cast crest on the ogee weir were recognised however, and the ogee weir was progressed as a combined pre-cast and cast in-situ option.

The access constraints also impacted the design of the in-situ sections. The remoteness of the site, and the likely duration between mixing and pouring the in-situ concrete, had to be considered within the concrete mix design.

Modifications to the existing structure

The concrete block below the existing gate structure, and the spillway downstream of it, are formed of mass concrete with a stronger shell covering a weaker core. The drawings noted that shell was formed from "Class B" concrete and the core was formed from "Class D" concrete. While the strength of these mixes can be inferred from historic design guides, the design was progressed on conservative assumptions.

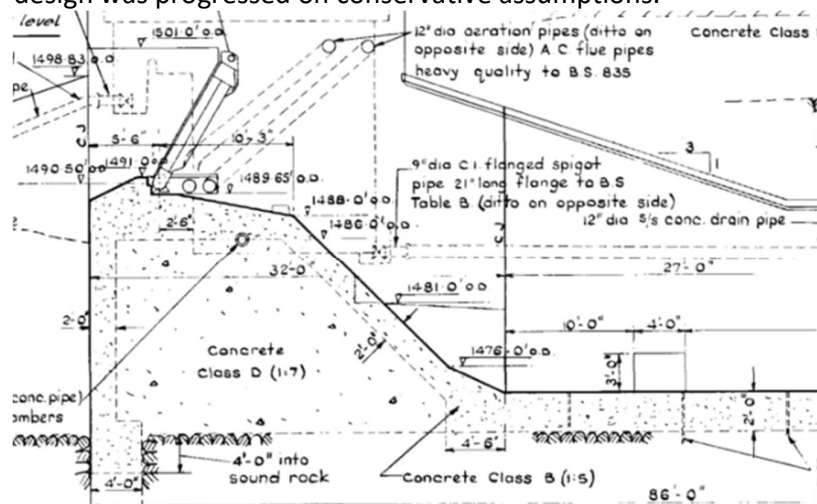


Figure 5. Cross-section through existing weir block

For the spillway channel, this weaker core did not present an issue, but for the weir block, the modifications to accommodate the construction of the new weir would necessitate removing much of the stronger shell, exposing the weaker core.

Initially, the weaker core was used as the foundation for the new weir, but the expected concrete strength drove a very onerous dowel design, and there were concerns over the long-term durability of the surface. Instead, a structural capping layer to the weaker core was designed to reinstate the stronger "shell" which was present in the original design. This solution simplified the dowel design and removed the durability concerns.

Optimising reservoir flood safety and storage capacity: Comparative design of labyrinth and ogee weirs

During the tender phase, cores were taken from both the stronger shell and the weaker core, and strength testing was undertaken to verify the design assumptions.

Cost comparisons

The initial view of the Reservoir Manager, and one which was supported by the Designer, was that the labyrinth weir option would prove to be more costly, and therefore, the magnitude of the cost variation compared to the ogee weir had to be quantified, such that a business case could be developed for selecting the more costly option in order to retain a greater storage. This logic is what drove the dual detailed design.

Once the tender phase had been complete, it became apparent that the labyrinth weir option, as viewed by contractors, was the less expensive option, rendering the need for the business case moot.

DISCUSSION AND CONCLUSIONS

The Liath Reservoir study highlights the challenges of managing reservoir safety and reliability of critical flow control structures. The existing tilting gate presents a significant residual risk due to uncertainty in its operation, particularly during extreme events and under site constraints such as limited access and exposure to severe weather. This justified the move towards a passive spillway solution that removes reliance on mechanical systems and improves robustness during flood conditions.

The design process demonstrates the importance of integrating hydraulic performance, storage capacity, and constructability in option appraisal. Both the ogee and labyrinth weirs provide compliant flood performance and eliminate the operational risks associated with the existing tilting gate. However, their performance differs significantly in terms of storage capacity, which is a key consideration for a hydropower dam.

The labyrinth weir offers a clear hydraulic advantage, passing equivalent flows at lower head and allowing a higher weir crest level than the ogee weir. This reduces storage loss to around 10%, compared to 27% for the ogee option, demonstrating how innovative geometries can limit operational impacts of safety upgrades.

However, site constraints strongly influenced feasibility. The remote location restricted construction methods, requiring the labyrinth weir to be cast in-situ, while the ogee allowed for a hybrid solution. This highlights the need to balance hydraulic performance with practical delivery considerations. Existing structural conditions also required adaptation, with

a capping layer introduced to address weaker core material and ensure durability.

Notably, tendered costs showed the labyrinth weir to be less expensive, challenging assumptions about complexity and reinforcing the value to the Client of detailed option development.

Overall, while both options are compliant, the labyrinth weir provides advantages in storage retention and cost, supporting a balanced, optimisation-led approach to spillway design.

REFERENCES

- Crookston B M and Tullis B P (2013a). Hydraulic Design and Analysis of Labyrinth Weirs. I: Discharge Relationships. *Journal of Irrigation and Drainage Engineering*, 363-370.
- Crookston B M and Tullis B P (2013b). Hydraulic Design and Analysis of Labyrinth Weirs II: Nappe Aeration, Instability, and Vibration. *Journal of Irrigation and Drainage Engineering*, 371-377.
- Institution of Civil Engineers (2015) *Floods and Reservoir Safety 4th Edition*. ICE, London, UK
- ICOLD (2016). *ICOLD Bulletin B172 Technical Advancements in Spillway Design*. ICOLD, Paris, France.



The British Dam Society

Welford Reservoir Venturi Primed Flexi-Hose Syphon - A UK First?

J M STANWAY, Canal and River Trust

SYNOPSIS

This paper covers the Welford Reservoir temporary syphon project, in which an alternative solution using a venturi-primed syphon on Bauer hosing was used to act as a temporary replacement for an old degrading outlet works. It was designed, installed and commissioned all within seven months. Impressively, the constructed syphon can feed the canal downstream multiple days at a time and has been designed to be easily adapted into a permanent syphon in the future.

The paper covers the background of the project, key elements of the conceptual and detailed design, construction, commissioning, operational use and key lessons learned.

INTRODUCTION

Welford Reservoir is an earth embankment dam located on the Leicestershire - Northamptonshire border. It was constructed in 1838 and has a capacity of 301,400m³. It acts in cascade to the upstream Sulby Reservoir (Figure 1), and its primary purpose remains to provide water to the Grand Union Canal. The Undertaker is the Canal and River Trust.

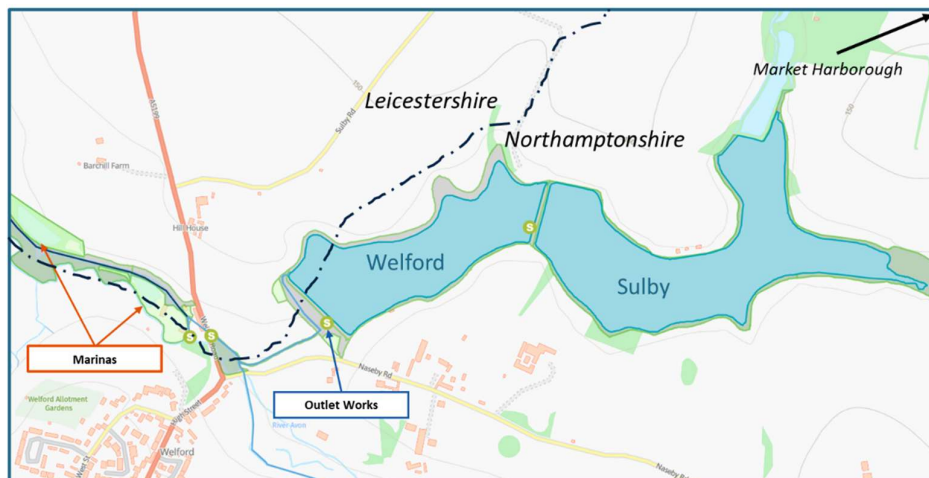


Figure 1. Map of Welford Reservoir

Welford Reservoir Venturi Primed Flexi-Hose Syphon - A UK First?

Following a partial blockage of the single outlet works the Section 10 inspection of March 2024 required the 'function and security of the outlet works to be restored with one year'.

Theories over the cause of the blockage ranged from a vanished buoy which previously marked the entrance to the outlet sinking into the culvert, to the identification of imperial sized bricks in the tailbay downstream, possibly being a sign of a decaying culvert with bricks falling and moving downstream.

INVESTIGATIONS

The original plan was to undertake a series of remote inspections to investigate the condition of the culvert and remove the partial blockage.

Two attempts using remotely operated vehicles (ROV) took place and failed due to poor visibility caused by heavy siltation on the upstream side of the outlet works. This included the use of a tracked ROV from the downstream side and a submersible ROV on the upstream.

Consequently, a change of plan was agreed with the Inspecting Engineer to install a new temporary outlet works structure with a flow rate either equal to or greater than the drawdown capacity of the old outlet works ($0.24\text{m}^3/\text{s}$). Works to reinstate the permanent outlet works are to follow under a separate project, including with a new and enlarged spillway facility.

All conceptual and detailed design work was completed in house by the Canal and River Trust, with Arcadis acting as Principal Designer.

CONCEPT DESIGN

Options

The primary consideration during concept design related to what type of temporary outlet works should be incorporated and whether it was more cost effective to procure or hire them. Options included:

- Installation of a temporary syphon via procurement
- Installation of a temporary syphon via hire
- Installation of a submersible pumping set-up via hire
- Installation of a mobile over-pumping set-up via hire

Whilst the pumping options had an initial cheaper installation cost, excessive annual hire costs of circa £150-220K (depending upon option) meant that if the temporary outlet works are to be retained for longer than two years, the temporary syphon via procurement option would become the most cost effective.

With there being no guarantee the original outlet works could be reinstated, it was decided that the temporary syphon would be designed with a long term solution in mind; that is a syphon which could be adapted to become a permanent outlet works either as a secondary outlet, or as the primary with the old outlet works being secured and abandoned.

Subsequently, it was decided the syphon procurement option was the best option. A significant advantage of this option was that a syphon only requires fuel for priming purposes. Once primed no further energy is required; whilst a pump requires constant fuel.

Syphon Layout

For the syphon layout (Figure 2), there is an upstream inlet, a priming area at the crest and a downstream outlet. There are three valves, one at the crest, one at the outlet and a smaller one to isolate the priming area. The upstream crest valve acts as the guard valve; it is not necessarily required but provides a fallback to turning the syphon off. The downstream duty valve is the valve which controls the syphon flow. There is no need for an upstream valve in the reservoir body when the system is venturi primed.

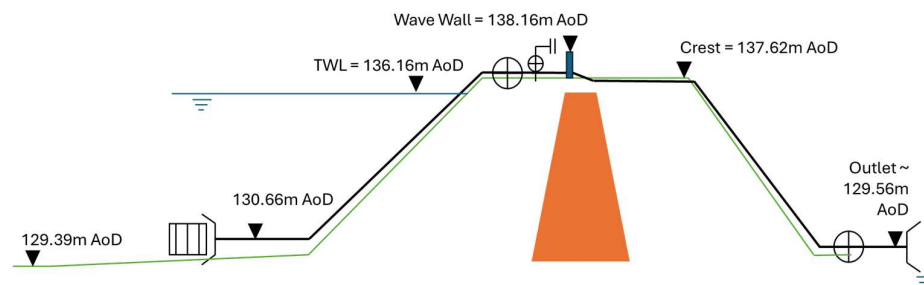


Figure 2. Sketch of the syphon layout

Syphon Routing

Due to the temporary nature of the syphon, it was agreed that the syphon should avoid going through the crest of the dam from the upstream side if possible. Such a layout would add significant complexities to the project leading to an increase in cost and duration. The original plan was therefore to break the syphon through the wave wall and build up a ramp above the syphon pipe to enable access to farmer's vehicles.

In terms of where to locate the syphon, there were three viable options:

- Route syphon into the spillway channel entrance > Discarded as would be difficult/dangerous to operate when the spillway is overflowing.

Welford Reservoir Venturi Primed Flexi-Hose Syphon - A UK First?

- Route syphon to follow the line of the left mitre into the tail-bay > Impossible to enable farmer's access to crest track as ramp cannot be built up in this location.
- Route syphon directly in the centre of the dam into the spillway channel > Chosen option (Figure 3).



Figure 3. Sketch of syphon route and items of note

Directing the syphon along the centre was the preferred option as it was the shortest length possible and it provided sufficient space for a ramp to be built up which the syphon could go through, and farmer's vehicles pass over.

In terms of length, on the upstream side the syphon needed to reach an operational range low enough to empty the reservoir to 10% volume. Whilst on the downstream side the syphon would empty into the spillway channel. Going along the centre of the dam, this arrangement led to the total length per syphon being 60m (Figure 4).

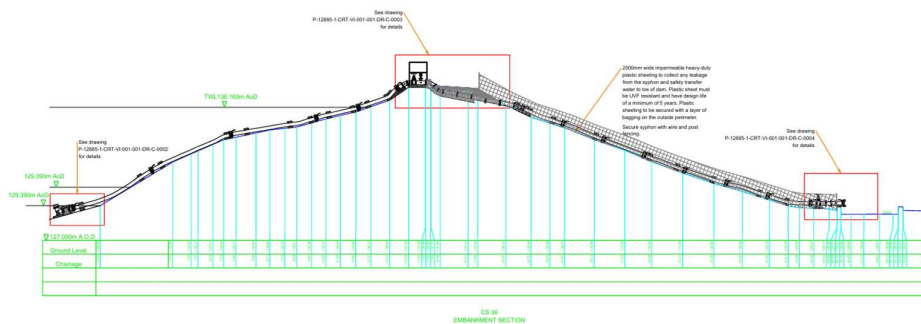


Figure 4. Longitudinal section of the syphon

Draw-off Flows

Taking the view of the syphon potentially becoming the primary outlet, the required draw-off rate must therefore follow the guidance from the 'Guide to drawdown capacity for reservoir safety and emergency planning'.

Stanway

The reservoir is Category A. The minimum recommended drawdown rate is therefore 5% H/Day; with a maximum retained reservoir depth of 6.8m this equates to 0.34m per day ($0.33\text{m}^3/\text{s}$).

However, with the outlet works also being used as a canal feeder the new works must also be capable of a reduced flow of $0.05\text{m}^3/\text{s}$.

Priming

With the Canal and River Trust already having two venturi primed syphons (Bosley and Knippton) and with the compressor units being readily available it was decided to use the same system at Welford Reservoir. This enables the Trust to share equipment, training and resources across the reservoirs and builds familiarity in how to operate the systems, improving the operational team's ability to react during an incident and to receive advice if anything does not operate as intended.

DETAILED DESIGN

Materials

For ease of construction 'Bauer' type flexi-hoses were chosen. At the inlet, priming area and the outlet ductile iron pipework fittings were used. This means that should the syphon ever be upgraded into a permanent set-up, the Bauer hoses can be replaced with ductile iron or polyethylene pipework. The Bauer hoses have sufficient UV grading to last up to 10 years in direct sunlight, so will last for the time required and, if necessary, be replaced easily.

The Bauer sections were originally proposed to all be in 6m length hoses. However, buildability discussions with the contractor led to the downstream hoses being in 3m lengths to reduce manual handling risks. The upstream remained in 6m lengths because these were floated in and sunk using divers.

Diameter

From the total length, geometric parameters and hydraulics, calculations for the pipe size needed to achieve the required draw-off rate was 250mm nominal bore. Alternative arrangements including a larger upstream diameter and smaller downstream were considered however, these achieved a too small flow rate. Specifically, calculations showed that a 250mm syphon would achieve a flow rate of $0.19\text{m}^3/\text{s}$, whilst for two syphons $0.38\text{m}^3/\text{s}$. This system would act as a duty-duty set-up, providing resilience should one syphon de-prime. The two smaller pipes also enable an improved turn down rate helping provision for the required canal feeder flow.

Welford Reservoir Venturi Primed Flexi-Hose Syphon - A UK First?

Another consideration was accommodating the pipework underneath a ramp. As pipe sizes increased this being more difficult. A pipe size of 250mm would require a ramp length of approximately 10m at a 1:8 gradient.

Crest Section

As the design developed, an alternative solution was realised by which the pipe would sink below the crest immediately after breaking through the wave wall. This would improve safety for the farmer's vehicles by removing the 'bump' across the crest. A 250mm pipe could just be accommodated in this arrangement. Any larger pipework would expose the pipework to the edge of the crest track and be vulnerable to being hit by a vehicle. By using a steel sheet above the pipework, the weight of the farmer's vehicles could be spread and the required cover reduced to a minimum on the wave wall side of the crest (Figure 5). This solution simplifies the ability of the syphon to be upgraded with permanent pipework by not requiring future trench works.

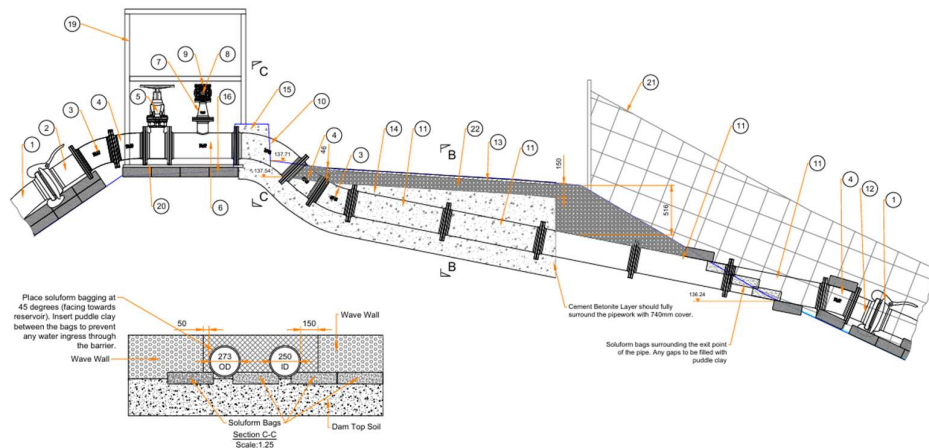


Figure 5. Syphon priming area and crest section

The gap in the wave wall would temporarily be blocked up using timber boards and soluform bags to cover any gaps. Once the crest pipework was installed this would be replaced by a concrete wall with waterbars surrounding the pipework.

There were concerns that as material was removed to form the trench for the syphon pipeline to go through and with the wave wall being temporarily compromised, a flow path could be created potentially causing internal erosion. To mitigate matters a cement bentonite trench at a ratio of 1:2 was installed surrounding the pipework. This gave an estimated shear strength

Stanway

of approximately 75-90KPA after 28 days, a complementary strength to the earth embankment itself.

To complete the crest section a galvanised steel fence surrounded the priming area with a small entrance gate and hand wheel security covers to deter inappropriate use of the valves.

Downstream slope

The downstream pipework was laid on a waterproof sheet surrounded by soluform bags to provide secondary containment. This was to ensure that should a leak occur, it would not cause erosion of the downstream slope.

Inlet and outlet sections

For the inlet, a duckfoot arrangement was chosen. This would be surrounded by a bed of soluform bags to help stability. Such an arrangement would prevent the inlet silting up, reduce the risk of erosion in the area and helped ensure its operability.

Meanwhile, on the outlet a series of short ductile iron bends were used to help direct the syphon flow further down the channel as opposed to hitting the spillway channel perpendicularly.

Venturi

A venturi jet exhaust is a 'gun' shaped metal ancillary which is connected to an air compressor on the shorter stub length side and to the syphon on the bottom (Figure 6). The high velocity of the air being forced through the 'gun' creates low pressure, providing a suction force. Over time this suction will remove the air from the pipework, creating a vacuum which then pulls water out of the syphon. The longer length side of the venturi is the exhaust; water jetting out of this indicates the system is primed.

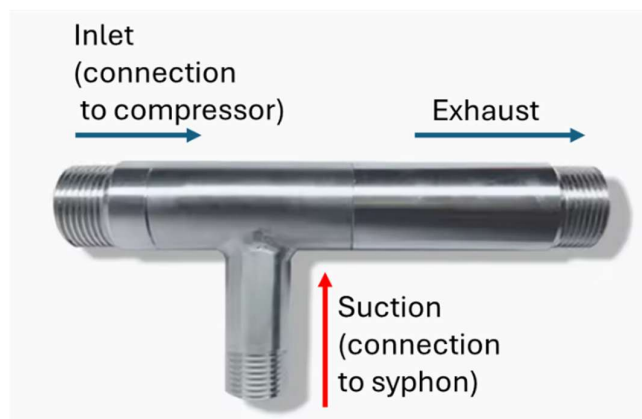


Figure 6. Venturi jet exhauster

Welford Reservoir Venturi Primed Flexi-Hose Syphon - A UK First?

To size the venturi the main consideration was to ensure the syphon could be primed within two hours of request. With manpower and transport issues potentially taking up to 90 minutes and set-up around 15 minutes; the syphon would have to prime within 15 minutes.

The Canal and River Trust had two compressors already in use – M13 and M20, producing 1.3m³/min and 2m³/min of air respectively. To remove the total volume of air in the pipework ~ 2.945m³, two venturi options were viable within the timescales available, a ¾" or a 1" venturi. With the M20 compressor being used alongside the 1" venturi, the expected priming time is around 4 minutes. With time of the essence this was the option chosen.

CONSTRUCTION

The Canal and River Trust had a framework contract with Kier, allowing them to procure their services on short notice. Early Contractor Involvement began in July 2025, and discussions continued through the following months.

Design began in August and was predominantly completed by the middle of November, with some aspects being re-designed whilst construction was taking place, notably the cement bentonite trench and the galvanised steel fencing around the priming area, which was a late addition to the design.

Kier mobilised in January, with trenchworks and pipe-testing taking place at first. The interim wave wall using timber boards was in place for about two weeks until the new crest pipework was laid and the new concrete adjoining section of wave wall was installed. The Bauer pipes and remaining ductile iron sections could then be laid on the upstream and downstream sections (Figure 7 and Figure 8). For all works in the reservoir itself specialist contractor Edward Diving Services (EDS) were used.

Including mobilisation, construction and de-mobilisation, the works took seven weeks in total with the syphon being successfully commissioned ahead of the MIOS deadline.

A construction phase on-site flood plan was in place during the construction project, meaning Kier was responsible for flood management issues related to their works.

There was an excellent collaborative relationship between the on-site team and the Canal and River Trust Project Team. This allowed any queries relating to the design or unexpected buildability issues to be dealt with at pace with solutions being discussed between both parties to find a pragmatic way forward.



Figure 7. The finished syphon (looking downstream)



Figure 8. The finished syphon (looking upstream)

COMMISSIONING

To commission the syphon, the compressor hose is connected to the venturi, which is connected to one of the syphons. All valves are opened and closed back to their original position to check their operability. The starting position for the upstream valve is open, the priming valve open and the downstream valve closed. The compressor can then be started up, and air will be forced

Welford Reservoir Venturi Primed Flexi-Hose Syphon - A UK First?

out of the pipework. As the air is removed, the water begins to be pulled out of the reservoir into the pipework. Eventually, water is jetted out of the exhaust; at this point the syphon is now primed. The priming valve must then be closed, stopping off the water jet action and then the downstream valve can be opened; a flow of water will now pass into the spillway (Figure 9). The same process can be followed for the second syphon.

To turn the syphon off the downstream and crest valves are closed. The downstream valve can be used to control the flow of water; if reopened the flow of water will begin again. Whilst the crest valve will control the ability of the syphon to hold prime; close this and the syphon will lose its prime and will require re-commissioning to start draw-off again.

Testing has shown the average syphon priming time when at/near top water level to be 3 minutes 45 seconds for the left syphon and 4 minutes 15 seconds for the right syphon (when looking downstream). The reason for this discrepancy could be due to air ingress in the right syphon, noted by noise at the Bauer pipework joints.



Figure 9. The outlet of the syphon during discharge

CURRENT OPERATIONAL USE

Upon testing, the syphon's actual draw-off flow rate was measured at $0.16\text{m}^3/\text{s}$ per syphon, $0.32\text{m}^3/\text{s}$ total for both; this is marginally below the minimum Category A requirements for draw-off ($0.33\text{m}^3/\text{s}$), but exceeds the previous outlet works flow of $0.24\text{m}^3/\text{s}$. Potential reasons for why the syphon flow is slightly lower than calculated include the depth of the inlet being higher and unaccounted hydraulic losses in the syphon system. Replacing the Bauer hosing with permanent pipework, with a less rough internal surface, would be expected to increase draw-off flow rate further.

The syphon is successfully being used as a canal feeder. By closing the downstream valve by circa 90% the flow rate is reduced to around $0.05\text{m}^3/\text{s}$. Tests were conducted showing that the syphon could be left on at this rate for $2\frac{1}{2}$ days before losing its prime. By fully opening the downstream valve every two days and giving the syphon time to regain full flow, approximately 5 minutes, evacuating any air that has been drawn into the system or come out of solution, the syphon can then be reduced back to canal feeder flow indefinitely when this sequence is followed.

In theory the syphon should not lose its prime if the velocity is high enough to evacuate any air coming out of solution, however when the downstream valve is either partially or fully closed and velocities reduced, there is a notable amount of leakage coming from the joints of the Bauer pipework – this may be due to these pipes being second hand (only items available). Over time this will lead to air entering the syphon and it de-priming. Fully reopening the valve (before the syphon has lost prime) appears to act as a flush, removing the air from the syphon and enabling it to regain full flow.

LESSONS LEARNT

Key lessons to be taken forward include:

- Engaging with remote inspection specialists early and being willing to progress with more costly/advanced techniques.
- Use of new Bauer hoses if available and/or replacing the seal to try and prevent air loss in the syphon system.

CONCLUSIONS

The venturi-primed Bauer-hosed syphon at Welford reservoir is potentially one of a kind. It is a cost-effective alternative to expensive pumping options and can be adapted for permanent use or re-used at alternative sites.

The project was completed on time and on budget. By going down the route of procurement, the Undertaker will have saved approximately £0.5M over

Welford Reservoir Venturi Primed Flexi-Hose Syphon - A UK First?

a five-year period. The application of using the syphon as a canal feeder has also prevented the need for a secondary syphon.

It can be concluded that the syphon has been a resounding success. Testament to this is by the fact the solution has since been proposed at other reservoirs within the Canal and River Trust portfolio.

REFERENCES

Department for Environment, Food & Rural Affairs & Environment Agency (2017) *Guide to drawdown capacity for reservoir safety and emergency planning*. Available at: <https://www.gov.uk/flood-and-coastal-erosion-risk-management-research-reports/guide-to-drawdown-capacity-for-reservoir-safety-and-emergency-planning> (Accessed: June 2026).



The British Dam Society

Core? What core? Lessons from Pre-1840 Embankment Dams on Locating Puddle Clay Cores

S BUTLER, Canal and River Trust
H FITZGIBBON, Mott MacDonald
N MILLER, Binnies
L STEDMAN, Mott MacDonald

INTRODUCTION

The Canal and River Trust has undertaken a full assessment of its portfolio of 74 regulated reservoirs utilising the EA Guide to Risk Assessment for Reservoir Safety Management (RARS) (EA, 2013). The initial assessment and scoping process described by Bennett (2022) has been fully implemented and scoping workshops have been undertaken for each reservoir to assess the relative proportionality of various risk reduction options. These range from small scale investigations and studies to large capital works. This continuous cycle of assessment, scoping, risk reduction and reassessment has been fully embedded across the Trust's business planning cycle.

In order to achieve risk reduction through investigation a project was set up to deliver geotechnical investigations and studies. The investigations were to provide the desired risk reduction but to also provide additional information to refine RARS scoring and add to the knowledge of the dam construction details. These types of investigations almost always were proportionate in terms of the risk reduction achieved.

Analysis of the most common failure modes across the portfolio indicate that internal erosion and buried structures provide the most contribution to the Annual Probability of Failure. This is not because these failure modes are inherently more significant but because as-built information is not readily available leading to conservatism.

This project identified reservoirs where some form of either Ground Investigation (GI) or study is required in order to confirm the RARS scoring given and provide the necessary risk reduction.

PRE-PENNINE ERA DAMS

The early dam building period in the UK (pre-1820) coincided with the canal era, when reservoirs were built primarily to supply water to canal network. Designs were empirical and based on experience and reliability improved through trial, failure, and repair. Embankments were constructed from

Core? What core? Lessons from Pre-1840 Embankment Dams on Locating Puddle Clay Cores

locally available materials such as clay, sand, and rock. The embankments were broad-based with steep slopes and simple internal zoning. A defining feature was the use of a central puddle clay zone: well-worked clay placed to form an impermeable barrier. While innovative, its success depended heavily on workmanship.

Rudyard Dam, constructed around 1799 to supply the Caldon Canal, is a representative example of early practice. It is an earth dam with a puddle clay core, built using local materials and simple construction methods. Its original spillway and outlet arrangements were basic and have been modified over time. Its continued operation reflects both the robustness of conservative embankment geometry and the importance of later improvements.

Previous authors have divided early UK embankment dams into the two broad categories of pre-Pennine and Pennine types (Moffat, 1999). Pennine embankments are characterised as having a central puddle clay core supported by mixed fill shoulder material. Pre-Pennine type dams are often categorised as having “no core” and described as homogenous.

The recent GI’s have found it is not always as clear cut as having a core or no core. Some dams thought to have no core show behaviour indicating a high hydraulic gradient across the dam not typical of a homogenous dam but with no clear distinction or difference in shoulder/crest soil properties. A summary of embankment properties encountered are given in Tables 1 to 4 below.

Table 1 Core Grading Properties

Dam name	Year	Proportion			Permeability
		Clay	Sand	Gravel	
		%	%	%	m/s
Rudyard	1800	11-35 [20]	15-36 [30]	4-43 [21]	10^{-8} to 10^{-9} [10^{-9}]
Slaithwaite	1797	13-63 [35]	11-60 [31]	0-76 [36]	10^{-11} to 10^{-9} [10^{-9}]
Toddbrook	1840	8-29 [20]	30-52 [46]	0-38 [4]	10^{-9}
Upper Foulridge	1866	12-35 [20]	20-25 [20]	1-6 [5]	10^{-8} to 10^{-9} [10^{-9}]
Lower Foulridge*	1798	6-37 [19]	7-52 [30]	2-62 [21]	10^{-4} to $<10^{-9}$ [10^{-6} to 10^{-7}]
Stanley Pool*	1840	15-32 [21]	22-40 [31]	4-21 [18]	10^{-9} to 10^{-7} [10^{-8}]

**Definitive core was not confirmed*

Table 2 Core Material Properties.

Dam name	Water Content w%	Consistency Limits		Plasticity Index	Liquidity Index	Classification
	w%	w _L %	w _p %	I _p %	I _L %	
	Range [Average]					
Rudyard	13.3-55.6 (17)	30-68 (38)	16-33 (20)	12-35 (20)	-0.22 to 0.65 (-0.15)	CL to CI
Slaithwaite	12-36 [22]	20-51 [24]	10-26 [19]	6-25 [15]	-1.2 to 0.88 [0.18]	CL to CI
Toddbrook	11.2-25.5 [17.7]	21-38 [27.9]	6.6-19.2 [12.9]	14-18 [15]	-0.8 to 0.73 [0.23]	CL to CI
Upper Foulridge	15-33 [26.7]	42-57 [51]	17-30 [23.5]	24-32 [27.7]	-0.125 to 0.222 [0.1]	CI to CH
Lower Foulridge*	7.1-35 [17.8]	20-56 [35]	10-29 [19]	9-31 [17]	-0.84 to -0.292 [-0.001]	CL to CI
Stanley Pool*	12-34 [18]	25-39 [35]	13-19 [16]	12-20 [16]	-0.083 to 0.75 [0.125]	CL to CI

*Definitive core was not confirmed

Table 3 Downstream Shoulder Grading.

Dam name	Year	Proportion			Permeability
		Clay	Sand	Gravel	
		%	%	%	m/s
Rudyard	1800	9-21 [15]	20-79 [25]	10-57 [33]	10^{-8} to 10^{-5} [10^{-7}]
Slaithwaite	1797	8-55 [24]	13-56 [27]	4-79 [49]	10^{-9} to 10^{-5} [10^{-6}]
Toddbrook	1840	2-12 [5]	16-95 [57]	0-59 [18]	10^{-4}
Upper Foulridge	1866	8-30 [10]	20-52 [30]	6-42 [25]	10^{-9} to 10^{-7} [10^{-8}]
Lower Foulridge*	1798	6-37 [18]	21-42 [34]	2-62 [26]	10^{-4} to 10^{-11}
Stanley Pool*	1840	0-22 [12]	16-31 [23]	19-67 [38]	10^{-8} to 10^{-4} [10^{-6}]

*Definitive core was not confirmed

Core? What core? Lessons from Pre-1840 Embankment Dams on Locating Puddle Clay Cores

Table 4 Downstream Shoulder Material Properties.

Dam name	Water Content w%	Consistency Limits		Plasticity Index	Liquidity Index	Classification
	w%	w _L %	w _P %	I _P %	I _L %	
	Range [Average]					
Rudyard	7.8-24.9 (20)	29-64 (35)	17-32 (21)	12-35 (15)	-0.767 to -0.203 (-0.067)	CL to CH
Slaithwaite	9.4-38 [19]	20-60 [38]	15-29 [21]	11-31 [18]	-0.36 to 0.82 [0.13]	CL to CI
Toddbrook	10-25 [17.2]	22-27 [24.7]	12.5-18 [15]	9-9.5 [9.1]	-1.6 to 0.42 [-0.17]	CL to CI
Upper Foulridge	10.3-37.9 [23.7]	33-46 [46]	18-28 [23.1]	9-24 [14.8]	-0.53 to 0.55 [0.07]	CI to CL
Lower Foulridge*	8.3-28.5 [16.9]	29-40 [34]	17-22 [17]	9-23 [14]	-0.84 to 0.33 [-0.190]	CL to CI

*Definitive core was not confirmed

INVESTIGATION METHODS

GI for older embankment dams is typically driven by the need to characterise the puddle clay, particularly where Measures in the Interest of Safety (MIOS) require investigation or RARS scoring updates requires confirmation of the core geometry, including the top elevation, width, and depth. Given the age of many puddle clay core dams original construction records are often incomplete or absent and intrusive investigation is required.

The use of non-invasive investigation methods, such as Geophysical methods like Electrical Resistivity Tomography (ERT), have been used on embankment dams to identify subsurface anomalies and variations in material properties (Park, 2017; Hamlyn, 2024). Geophysical survey results can be useful to identify variations, but on older dams there is usually little change in the material. In practice, the geophysics survey results are typically accompanied by caveats of the findings' interpretation, and for definitive results, an intrusive investigation would be required. On this basis, progressing to direct intrusive investigation was considered more appropriate on the following sites, avoiding additional costs and programme delays with additional survey phases.

The following sub-sections describe the intrusive investigation methods used to identify the top and width of the clay core.

Hand pits

Where the puddle clay core is suspected within the first metre below the crest, hand dug trial pits provided a direct low-risk method of investigation. Hand pits typically 0.6m wide are excavated to a maximum depth of 1.2m below ground level in accordance with industry practice for unsupported hand excavation.

To avoid creating a continuous line of disturbance across the embankment crest, successive pits are staggered to the left or right along the crest 1.2m or greater. This staggering arrangement ensures that when backfilled and reinstated, the pits do not allow a linear defect that could compromise the integrity of the crest.

During excavation, the soil profile is slowly exposed and logged by a geotechnical engineer, with particular attention to colour and soil consistency. When the clay core is found, typically a Disturbed 1l tub sample is obtained, and the excavation is stopped once the top of the core is reached, then extended wider to confirm the upstream or downstream extent, keeping within the pit maximum dimensions. If the upstream or downstream face is not found the next staggered pit is started. The method allows multiple pits to be opened simultaneously to measure the total width of the clay core.

Pits were backfilled with the excavated material, compacted in layers by hand tamper, and the crest reinstated to its original profile.

Slit trenches

Where a core is suspected within the first metre below crest level, hand dug slit trial trenches along the embankment crest to a maximum of 1.2mBGL were found to be a valuable investigation method. The trial trenches were split in two halves, offset from one another across the crest, to try and visually identify the clay core without compromising the integrity of the potential clay core. During and after excavation, modified soakaway tests were carried out in small pits adjacent to the trenches to measure the relative permeabilities of different fill materials and collect evidence for a less permeable core section. The main benefit of this method was the ability to visually compare the ground conditions across the trench and note any differences in colour or percentage of granular particles of the soil.

Core? What core? Lessons from Pre-1840 Embankment Dams on Locating Puddle Clay Cores



Figure 11 : Modified soakaway hand dug pit at Lower Foulridge.



Figure 22: Excavation of a slit trench at Lower Foulridge.

Window sampling

For cases with historic embankment raising without core raising, an array of window samples was used to try and identify whether a core was present. This method was found to be more challenging to identify the presence or otherwise of the core, as ability to visually compare the ground was lost. However, the method produced useful samples for laboratory testing and permeability testing could be completed in the hole. For each row in the array, the window samples were undertaken according to the sequencing shown in Figure 3.

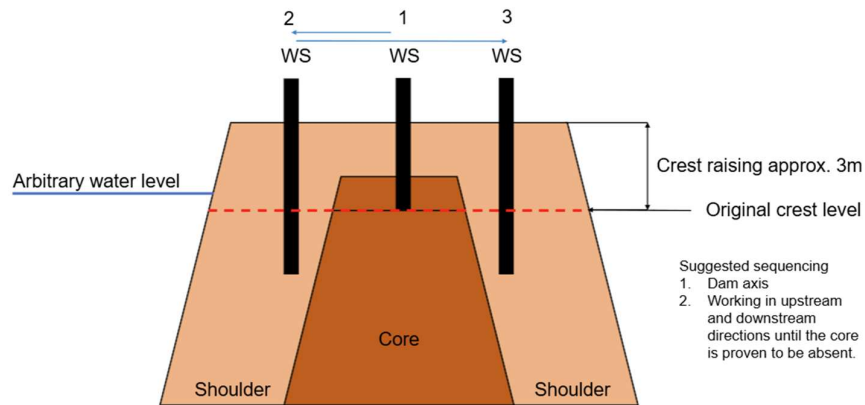


Figure 33: Schematic explaining proposed window sample sequencing.

CASE STUDY 1- LOWER FOULRIDGE

Introduction

Lower Foulridge is an impounding reservoir formed by the construction of an earthen embankment dam in 1798 to supply water to the Leeds and Liverpool Canal. The embankment has been subjected to three episodes of crest raising in 1814, 1832 and 1994 to a total increased height of 2.84m.

Ground Investigation

A series of GIs have taken place at Lower Foulridge since 2008, including investigation through the spillway and the base of the reservoir. GI was undertaken to ascertain the composition of the embankment and confirm or negate the presence of a clay core. This included cable percussive boreholes to a maximum of 17.8mBGL on the downstream shoulder through the embankment, or directly through the crest to try and locate the clay core and confirm the embankment-founding material interface. In addition, hand dug slit trial trenches along the embankment crest to a maximum of 1.2mBGL with soakaway testing, and hand dug trial pits around the outlets.

Further GI was undertaken comprising dynamic sampling boreholes to a maximum depth of 10.2mBGL, hand dug trial pits to a maximum depth of 1.2mBGL, and hand dug trial trenches along the crest to a maximum depth of 0.8mBGL.



Figure 4 - Mott MacDonald GI at Lower Foulridge Reservoir

Key Findings

The results of the GI concluded that Lower Foulridge should be characterised as a homogenous embankment, with the material found to be typically a “soft to firm grey to greyish brown or orange slightly sandy to sandy slightly gravelly CLAY with occasional pockets of SAND”. Based on the available GI data, there was no noticeable difference (visually or in laboratory test result) between the material at the dam axis and that on the downstream shoulder to support the presence of a defined clay core; it is likely that both were constructed from the same source material of Glacial Till.

The following evidence was gathered: 1) PSD grading curves do not indicate a trend in fines/gravel content through the embankment cross section- it could be expected that the downstream shoulder would have a higher

Core? What core? Lessons from Pre-1840 Embankment Dams on Locating Puddle Clay Cores

proportion of gravel compared to the core. 2) Index properties indicate low to intermediate plasticity with no distinct difference between the downstream shoulder and dam axis- it would be expected that the clay core would have intermediate to high plasticity. 3) Liquidity index of the fill at the dam axis and on the downstream shoulder overlap with no clear difference between the two, suggesting a different construction methodology was not used to create the central core zone, i.e. addition of water for puddling. 4) Piezometers show modest hydraulic gradients with limited response to reservoir levels, indicating low permeability, and no clear zoning.

The targeted GI on Lower Foulridge highlights the difficulty in identifying puddle clay cores within the field setting, emphasising that even with extensive laboratory testing, significant analysis is required to determine whether a clay core is present, or whether the dam is in fact homogenous. Here, Lower Foulridge was found to be an example of a pre-Pennine embankment with a homogenous clay section and a modest hydraulic gradient, as described in Moffat (1999).

CASE STUDY 2- SLAITHWAITE

Introduction

Slaithwaite Reservoir was constructed in c.1797 to supply the Huddersfield Canal. The reservoir is underlain by the Marsden Formation of the Millstone Grit Group. No superficial deposits are mapped directly beneath the reservoir.

Construction History

The reservoir was constructed in c.1797 and was reportedly 'made watertight' in 1799 due to persistent leaking. In 1799, unprecedented floods required immediate action to cut the embankment open in two places to prevent failure. In 1803, concerns of settlement were reported following breakage of the main outlet, and in the early 1970s, major spillway modifications were undertaken, including extension of the weir, lowering the invert, and widening the upstream end of the channel. Further remedial works were undertaken in 1991/1992 where the embankment was raised by approximately 0.7m.

Ground Investigation

A series of GIs has been undertaken at Slaithwaite since 1974. A seepage toolbox assessment for Slaithwaite was undertaken to investigate the susceptibility of the embankment to seepage and internal erosion, with GI undertaken to ascertain the composition of the embankment. This included

dynamic sampling with rotary core follow on to a maximum depth of 24.2mBGL and concrete cores through the spillway with dynamic sampling/rotary core follow on to a maximum depth of 2.9mBGL.

Key Findings

The results of the GI concluded that Slaithwaite has selectively placed fill which has been puddled into a 'core zone'. The material forming the core was found to be gravelly sandy and silty clay, clayey sands, and sand-clay mixtures of low to intermediate plasticity. The downstream shoulder material comprised similar material; however, the key differences lay in the lower proportions of fines (average 24% compared with 35% in the core), and higher proportions of gravel (average 49% compared with 36% in the core) in the downstream shoulder, as in Tables 1-4. Other distinct differences between the core and downstream shoulder fill include lower permeability of the core (average 10^{-9} m/s compared to 10^{-6} m/s in the downstream shoulder), and a non-suffusive core versus a suffusive downstream shoulder.

CASE STUDY 3- RUDYARD

Rudyard Reservoir, introduced above, is of particular significance to this paper. The widespread adoption of puddle clay cores in British dams is generally associated with the Pennine era, 1840 onwards (Moffat, 1999). The specification prepared by John Rennie in 1797 includes a clay core some 40 years before clay cores were thought to be widely adopted, and is considered one of the earliest surviving documented specifications with explicit construction methodology for an embankment. It has been described as showing "innovation in its approach to maintaining wet conditions within the clay core during construction" (Langford, 2021).

The specification described the embankment as an arch form bowing upstream in plan, 36ft high with a 40ft crest, and a puddle clay core 12ft thick at the base and 6ft at the top, founded on solid impervious ground across the valley floor and abutments. The core material was to be "good clay or clay and gravel mixed in the proportion of about 2 of the former to 1 of the latter", placed in layers approximately 8 inches thick, each covered with water and worked until saturated. The embankment fill on both sides was to be shaped to lean towards the core and placed in layers no greater than 3ft thick.

The GI to find the clay core used window sampling on the embankment crest, as the core was expected to extend below the practical limit for hand excavation. The crest at Rudyard up to 19m wide at the centre of the embankment crest, narrowing towards the abutments, and this made

Core? What core? Lessons from Pre-1840 Embankment Dams on Locating Puddle Clay Cores

locating the core with individual sample holes difficult. Several holes were required before the core position was confirmed. The core was also found to widen towards the centre of the embankment.

The puddle clay was identified visually in all holes. The core material was grey to greyish reddish brown and homogeneous, while the shoulder fill was reddish brown and contained sand lenses. This distinction was consistent in all holes and at depth.

KEY CHALLENGES

There are several challenges associated with identifying puddle clay cores and carrying out GIs on old embankment dams. The following sections briefly outline these challenges.

Drilling methods

Crests on old dams are typically narrow with steep slopes either side, restricting rig size. Smaller, lightweight rigs can be the only choice but typically achieve poorer recovery and are limited in the depth they can reach, possibly falling short of the base of the core. Where embankments have been raised, the core may sit close to the upstream or downstream slope rather than beneath the crest centreline, and locating it can require borehole positions at the crest edge or on the slope itself. In these cases a temporary working platform may be needed to provide safe and level standing for the rig, adding cost and programme to the investigation.

Defining the core

Defining a clay core within old embankment dams presents several challenges. Later crest raising following the embankment construction can obscure the original core profile creating an unusual dogleg within the core profile which is often hard to distinguish during a GI, particularly if boreholes are positioned through the centre of the embankment crest.

A further challenge in identifying the clay core arises where the material properties of the core and adjacent shoulder fill are similar, so that little distinction, both visually and through in-situ testing, can be made between the two during the GI. As a result, laboratory testing is required to identify any differences in particle size distribution, plasticity and so on, but even still, laboratory testing may not reveal a conclusive clay core. This was the case for Lower Foulridge.

Finally, some clay cores may extend beyond the reach of shallow investigation methods, such as dynamic sampling. Deep cores can therefore

be difficult to identify or be easily mistaken as a homogenous if it appears to extend for the entire depth of the embankment.

General challenges

Beyond the difficulties of locating and identifying the core, the investigation must not compromise the dam. Every intrusive hole is a potential defect, and the value of the information gained must be weighed against the disturbance caused. For hand-dug pits and slit trenches, excavation should not reach the reservoir water level, as an open excavation at this level risks creating a seepage pathway during the investigation. Reinstatement is as important as the investigation: holes and pits must be backfilled and sealed to ensure no preferential flow paths remain.

CURRENT GUIDANCE AND MODERN INVESTIGATION TECHNIQUES

UK guidance on investigation of embankment dams remains BRE 303 (1996) and BRE 363 (1999), both now archived. Both predate the availability of techniques that are now standard or near-standard on overseas dam investigations and increasingly used in the UK.

Suggested future considerations for GI in embankment dams are:

- Drilling. More recent drilling techniques such as Sonic and GeoBoreS are being used in the wider UK GI market. Sonic drilling is used increasingly in dam investigations in North America because it provides continuous casing, good core recovery, and requires less water flush compared to conventional rotary (USBR, 2014; FERC 2016).
- In-situ testing. CPT/CPTu provides continuous profiling with minimal disturbance, and seismic CPT (SCPT) adds shear-wave velocity for stiffness and liquefaction assessment. The controlling concern for CPT on dam crests is reliable sealing of the hole on withdrawal. Grout on withdrawal tips have been developed to address this, but enquiries made during this paper preparation indicate the capability is not yet readily available in the UK market. There is, however, interest from the UK CPT industry in developing reliable backfilling methods, and demand from dam owners and panel engineers may accelerate this.

CONCLUSIONS AND RECOMMENDATIONS

The three case studies show that pre-1840 embankments do not divide neatly into dams with a core and those without. Lower Foulridge was found to be homogeneous, contradicting the earlier assumptions of a core. Slaithwaite has a puddled core zone, but it could only be confirmed through

Core? What core? Lessons from Pre-1840 Embankment Dams on Locating Puddle Clay Cores

laboratory testing because the core and shoulder were won from the same source material. At Rudyard, the core was distinguishable on site. All three dams were built within a few years of each other, and the differences in how readily each core could be identified may stem from the available borrow materials and the quality of the puddling rather than the age or type of dam.

UK guidance on the investigation of embankment dams predates 25 years of development in investigation techniques. Updated published guidance, drawing on overseas practice and the methods now routinely used by UK practitioners, would benefit owners, panel engineers and designers scoping investigations on embankment dams.

REFERENCES

- EA, (2013) *Guide to risk assessment for reservoir safety management*. Environment Agency, Bristol, UK
- Bennett N, Foster J and Hewitt M (2022) RARS – A Client’s Perspective. In *Dams and Reservoirs in a Climate of Change – Proceedings of the 21st Biennial Conference of the British Dam Society*. British Dam Society, London, UK. pp 41-52
- Moffat A I B (1999). Puddle Clay and the ‘Pennines’ Embankment Dam. *Dams and Reservoirs*. **9(1)** pp 9-15
- Park C (2017). *Geophysical methods for reservoir safety investigation*. Commissioned by the Environment Agency and British Dam Society,
- Hamlyn J E, Bird C L, Cooper M J, Cottrell R J and Hayward M (2024). Looking into reservoir geophysics – Emerging technologies. In *Managing Risks for Dams and Reservoirs – Proceedings of the 22nd Biennial Conference of the British Dam Society*. British Dam Society, London, UK
- Langford (2021) *The development of early reservoir to supply water to arterial canals in England and Wales*.
- BRE (1996). *Report 303. Investigating embankment dams: a guide to the identification and repair of defects*. Building Research Establishment, Watford, UK
- BRE (1999). *Report 363. An engineering guide to the safety of embankment dams in the United Kingdom. 2nd edition*. Building Research Establishment, Watford, UK
- USBR (2014). *Guidelines for Drilling and Sampling in Embankment Dams*. United States Bureau of Reclamation, Washington DC, USA.

Butler et al

FERC (2016). *Guidelines for Drilling In and Near Embankment Dams and Their Foundations*. Federal Energy Regulatory Commission, Washington DC, USA.



The British Dam Society

Witley Court: Incident Response and Interim Repair

S HOLDEN, Mott MacDonald

T WEBSTER, Mott MacDonald

A WARREN, Mott MacDonald

SYNOPSIS

Witley Court is a historic ruined house and gardens operated by English Heritage. The estate includes an 18th century ornamental lake that falls under The Reservoirs Act 1975. The reservoir is impounded by an earthfill embankment dam at its east end, which has records of remedial works dating back to the late 1980s. A new void was recorded on the downstream face by the Supervising Engineer in October 2024, triggering an S10 inspection. Indications were that the void was caused by internal erosion due to leakage. Subsequent safety recommendations included an interim repair, undertaken in Spring 2025.

The interim repair comprised a programme of void filling by grouting from the crest to ensure safe access was maintained. During the works, a sewer pipe, previously thought to be abandoned, was found to be damaged and actively discharging water into the void. This changed the understanding of the failure mechanism from internal erosion to direct wash out. The sewer system was traced upstream to its source and bunged to prevent flow onto the embankment. The downstream, broken end of the pipe was sealed before the void was fully backfilled.

Further safety works are planned to strengthen and stabilise the left-hand side of the embankment and spillway.

INTRODUCTION

Witley Court was a vast country house that was devastated by fire in 1937. Due to prohibitive repair costs, Witley Court fell into ruin and the estate passed through several private owners until the house and gardens were transferred to the stewardship of the newly formed English Heritage in 1984, who have since undertaken restoration of the gardens to their present condition. The house remains as a vacant shell and is uninhabitable. The gardens include Witley Court Reservoir, also known as Front Pool, a 70,000m³ ornamental lake that was constructed in the mid-18th Century and lies to the north of the house (Figure 1).

Witley Court: Incident Response and Interim Repair



Figure 1. Overview of the dam embankment and reservoir. The overflow and outlet works are located at the left abutment (right side as viewed).

DESCRIPTION OF THE RESERVOIR

Witley Court Reservoir is orientated northwest to southeast with the embankment impounding the southeastern end. The earthfill embankment is 11m high and 95m long with a current crest level of 100.9 metres above Ordnance Datum (mOD). The embankment extends at its upstream mid-point into a semi-circular promontory, and a masonry wall extends the full length of the upstream crest at and below the waterline. The gradient of the downstream face is typically 1:2.5, with significantly steeper sections adjacent to the spillway at the left-hand side. The top water level of 98.9mOD is defined by the overflow channel, comprising a 4m wide concrete slab with masonry and stone-dressed concrete side walls, terminating above a vertical blockstone cascade wall and plunging some 6m to a sandstone stream bed in a gorge at the left abutment (Figure 2). Upgrade works undertaken in 2003 added a 2m wide by 1.4m high rectangular reinforced concrete auxiliary spillway to the left of the main spillway with a weir level of 99.1mOD, also discharging as a cascade immediately downstream of the main spillway. Additional works at this time included a new stone-dressed reinforced concrete pedestrian bridge with increased clearance height over the main spillway invert.



Figure 2. Overflow channel in operation as a cascade.

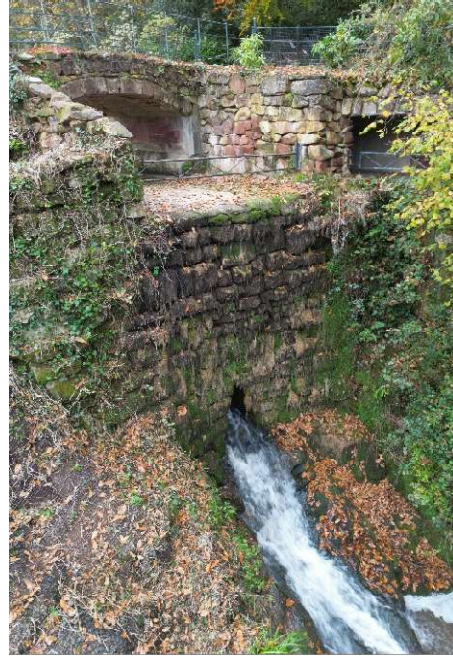


Figure 3. Outflow from the outlet chamber below the spillway cascade

The outlet works at Witley Court Reservoir are unusual, comprising a penstock gate over a 350mmØ bell mouth pipe, which drops vertically then turns through 120° to discharge into 7m long 500mmØ brick lined culvert. The culvert leads to a stone masonry outlet chamber with an arched brick roof, approximately 2m long, which in turn drops into a much larger chamber, approximately 4m by 4m in plan and 3m high. The floor and left side of the chamber are formed of natural rock, while the other side walls are of stone masonry construction. Flows outfall through a 1.8m long opening in the east side of the chamber, fashioned to resemble a natural grotto at the base of the spillway cascade wall (Figure 3), and discharge to an entrained channel flowing southwest away from the embankment toe. The outlet works allow the reservoir to be lowered to 1.6m below top water level.

HISTORIC MODIFICATIONS AND REMEDIAL WORKS

Due to the age of the asset and where ownership of the estate had changed hands multiple times through the 20th Century, there was no reliable narrative for the reservoir and embankment prior to the first Section 10(3) report under The Reservoirs Act 1975 in 1989.

The first inspection made note of ‘considerable leakage through the dam adjacent to the spillway wall’. The leakage was traced at the time to a

Witley Court: Incident Response and Interim Repair

fractured drainage pipe traversing the length of the dam, which had 'washed out a considerable amount of fill'. The action taken at the time was to renew the pipe and backfill the voided area with suitable material. It was recorded that the leakage was not completely sealed, and a v-notch was installed to monitor for any change in leakage rate and for any wash-out of fines.

Subsequent inspections by the Supervising Engineer noted the leakage was equivalent to a 20mmØ pipe running at full bore. Deposits of yellow clay were noted in the v-notch the following year, and a gravel pack filter was installed to try and reduce the wash-out. By 1990, flow over the v-notch was reported as having doubled, but a reduction in the reservoir water level caused the leakage to stop completely, suggesting the ingress point was within the top 180mm of normal water level. Repointing of the headwall masonry was undertaken, and leakage ceased completely – the v-notch was subsequently dismantled and backfilled with granular material after 1992.

The 1998 S10 inspection noted seepage through the right-hand wall of the outfall channel emerging downstream of the cascade, in an area of instability, and instructed rebuild of the training wall at the toe of the dam with drainage and granular fill. The auxiliary spillway was designed and constructed in response to other recommendations of this report. In addition to the spillway works, a new footbridge over the spillway was constructed to increase the clearance above the invert, and the crest level of the embankment was raised by up to 1.6m to the current uniform level of 100.9mOD, to tie-in to the new bridge level.

The 2006 S10 again noted the seepage through the right-hand wall of the spillway channel in the same location as previously noted. Damp areas around the toe and downstream face were also noted, along with desiccation cracks, animal burrows, and a bulging area not previously observed. Site investigation works were instructed to install monitoring apparatus and establish the parameters of the dam and foundation soils. Following the investigation, a 2m wide berm was constructed across the right 2/3 of the embankment in 2007 and a new right-mitre and toe-drain were installed, with a v-notch chamber at the toe to monitor leakage rates. Later that year, regional flooding resulted in erosion at the toe of the embankment. This was remediated by installation of gabion baskets supporting coarse granular infill material.

By 2015, the seepage downstream of the cascade was seen to be increasing along with deposition of sand and silt into the spillway and outlet. A 2016 'Willowstick' geophysical survey identified preferential flow paths through the dam's right abutment and below the spillway chute. A large package of remedial works was undertaken later that year to seal the upstream end of

the spillway with a mass concrete trench, repoint the spillway masonry and extend the spillway invert upstream with mesh reinforced concrete. The upstream masonry wall was repaired with blockwork and stone pitching. Instability in the downstream face adjacent to the spillway was reinstated with Type 1 fill and geogrid reinforcement, while the channel training wall was repointed and fitted with seepage monitoring tubes. Beyond the training wall, the embankment toe was buttressed with gabion baskets and rockfill. The right-hand mitre trench was intercepted with a new filter trench discharging directly to the downstream watercourse.

BACKGROUND TO RECENT WORKS

During routine grounds maintenance in October 2024, a substantial void was found near the crest on the downstream face of the dam embankment, immediately adjacent to the spillway right-hand wall and extending below the crest footpath (Figure 4). The Supervising Engineer was notified and, due to the voiding representing a serious reservoir safety incident, he called for a Section 10 inspection.

The main area of voiding was located adjacent to the right wall of the overflow channel and just downstream of the dam crest. Though the extent of the void could not be safely determined, the base was at least 2.5m below ground level. A clay pipe opening was noted passing below the spillway slab, found to be dry and understood to no longer be in service. A concrete slab could be seen near the roof of the void on the dam crest side, which appeared to be part of a mass concrete strip foundation installed during the 2003 works and which may have prevented collapse of the crest path. There was also evidence of minor settlement of the crest path surfacing to the right of the bridge.

Further down the embankment face was a larger area of depression with a depth of up to 1m below the surface reinforcing geogrid mesh, with anchoring pins no longer embedded in soil, demonstrating a significant erosion of material.

The subsequent report following the inspection included a number of recommended measures in the interest of safety (MIOS), including:

“An interim repair... to the voiding under the dam crest path to restore safe access across the crest.”

The left side of the embankment was considered to be of poor inherent design, being over-steep. Given the patchwork nature of historic remediation and the deterioration of the left side of the embankment, major works to improve the safety of the dam were also recommended in the shape of investigations, design and construction works to remove and

Witley Court: Incident Response and Interim Repair

reconstruct the damaged earthfill to improve slope stability and resilience to seepage and erosion.



Figure 4. Daylighting void adjacent to right spillway wall, downstream of crest. Primary voiding circled red, secondary depression circled blue. Flows from the outlet chamber 'grotto' can be seen entering the streambed.

Following the issue of the S10 Inspection Report, Mott MacDonald Ltd (the Designer) was commissioned by the English Heritage Trust (the Client), to develop options for the interim repair, and to direct the implementation of remedial works for the chosen option. The interim repair was specifically intended to make safe the crest access path, as a closure would significantly limit access to the house and gardens both for groundskeeping staff and visiting members of the public. However, it was acknowledged that these works were not to be seen as seepage prevention works, rather underpinning of the path and risk reduction. A timeline of three months was given to complete the MIOS works for the interim repair, with the reservoir levels held down by over pumping in the meantime.

EROSION MECHANISM

Given the history of leakage at the dam and the conditions observed, it was assumed that the void was the result of ongoing internal erosion processes. Although the dam largely has a broad crest and shallow downstream slope, due to the sudden change in elevation adjacent to the vertical cascade, the steep downstream slope results in a locally high hydraulic gradient at the left

side of the embankment. It is also located at the interface between a hard structure and soil, where the embankment directly abuts the overflow channel and outlet 'chamber' with no shear key detail. Furthermore, the downstream embankment was previously vegetated with trees and shrubs that have since been cleared, and degrading root systems can leave voids and preferential pathways for seepage.

Previous remedial works had failed to adequately control erosion from the ongoing seepages, notably the use of coarse granular backfill not being sufficient to prevent wash-out of fines. It was also considered that inadequate compaction of backfill to the spillway bridge works may have created a zone of higher permeability in the affected area.

Normally, a void of this size would have been detected earlier as slumping on the line of the crest path. However, where the void passed below the concrete slab the surface had remained supported in place. This also would have reduced the vertical stress on the soil below the slab and potentially increased the rate of erosion, until such a point where the void collapsed in on itself to daylight on the downstream face.

Progression of a void towards the upstream face drastically shortens the seepage path, and if allowed to develop further risked a potential outburst breach of the reservoir. The dam has been assigned as Category B in accordance with Floods and Reservoir Safety, 4th edition (ICE, 2015) meaning that lives could be endangered in the event of a breach.

INTERIM REPAIR DESIGN

Limited information was available for the embankment following the 2006 ground investigation. It was determined that the embankment fill was a slightly sandy clayey silt (>80% fines). The presence of a clay core had not been investigated and given the provenance of the embankment was not anticipated to be present. Piezometers installed in section across the left-hand side showed an upstream water table in continuity with the reservoir level, falling to 3-4m below ground level (bgl) on the downstream side of the crest before meeting the stream at the toe. A very soft band of fill was recorded at 6m bgl, with the embankment fill being otherwise recorded as firm clay.

Options proposed for the interim repair were assessed on the basis of programme length, capital cost, flood risk and robustness to help the client make an informed decision (Table 1).

Witley Court: Incident Response and Interim Repair

Table 1. Shortlisted interim repair options

Option	Description
Excavate and backfill	Excavate failed material and dismantle the retaining wall supporting the crest footpath. Reinstall with compacted selected fill and reconstruct footpath supporting structure.
Polyurethane Resin Injection	Install steel lances and inject semi-flexible PU resin as a void filler across the treatment area in a grid pattern. Delayed reaction time ensures penetration into voids and fissures before expanding up to 20x volume.
Combination Grouting	PU Resin injection to upstream embankment only to decrease permeability. Drilling and bentonite cement grouting from the crest to target voids across a larger depth range.

Ultimately the combination grouting option was selected. This was primarily due to the lower flood risk compared to a dig and replace solution, with the extent of voiding and therefore maximum excavation unknown and potentially leading to significant programme extension and disruption. The material costs were also lower for bentonite cement grouting than PU resin injection, given the potential volumes required. Bentonite cement grouting could also be applied over a deeper treatment range than the PU injection, as it would not be limited by the maximum depth of hand inserted steel lances.

One of the key constraints for the proposed intervention was access. With the crest footpath being deemed unsafe for pedestrian access, movement of construction plant across the treatment area would not be permitted. It was agreed that access would be from the left abutment, where a drilling rig would be able to sit upon the mass concrete backing to the bridge arch and drill into the voided zone without bearing upon it. Following insertion and initial curing of grout, the rig could progress sequentially to the right of the bridge as the void was treated. Trial pits were scheduled to positively identify the extent of the bridge concrete before drilling works commenced.

INTERIM REPAIR CONSTRUCTION

The interim repair works commenced with Spencer Quantum Ltd (the Contractor) undertaking investigation trenches to locate the edge of the concrete bridge structure. In advance of grouting works, caulking of known seepage points on the downstream face and channel retaining wall were undertaken with sandbags and mortar.

The Contractor installed a staggered double row of 2m long lances at 1m centres, arranged in an L-shape immediately behind the upstream crest and

to the right of the spillway channel. Due to the proximity to the water body, the resin product selected would become chemically inert upon contact with water, to avoid the risk of any pollution in the event of escape from the upstream masonry wall. Resin was injected to each lance by electric pump until refusal, and the lances were withdrawn by 0.5m at a time with injection repeated at each horizontal level.

No volumes of resin higher than expected were injected, suggesting no significant voiding was encountered in this area. The resin product expands on contact with moisture, which has the effect of compressing the soil around each injection point and reducing the permeability of the treated area.



Figure 5. Polyurethane resin injection at the upstream crest to reduce permeability across the presumed infiltration zone for seepage.

Following the resin injection, the first grout hole was drilled immediately to the right of the bridge structure. This hole was progressed down to the embankment formation to terminate on the sandstone bedrock at 7.9m below crest level, retrieving samples for geotechnical testing and to inform design of the permanent solution. The drill hole was backfilled by tremie pipe from the base, removing casing in lifts so any voided horizons could be easily identified. The first drill hole did not intercept the void or any seepage paths and grout levels held. Advancing to the next location, the void was encountered at 1.3m depth below a zone of mass concrete infill associated with the retaining wall supporting the footpath. The hole was advanced to 5m depth with no deeper voids encountered before backfilling commenced with a 2:1 bentonite cement slurry. The grout was topped up multiple times as it flowed into the void and seepage pathways.

Witley Court: Incident Response and Interim Repair

During grouting, the Contractor alerted the Designer that a significant volume of apparent foul water was entering the void from a broken PVC pipe now visible entering the void from the right side. The flow continued for at least 10 minutes and with enough force to wash out some of the grout already placed. Works were halted immediately and an investigation launched into the source of the flows.

Service tracing undertaken prior to mobilisation had not identified any live services running across the dam embankment. The presence of an historic sewer pipe was referenced from previous inspection reports, but this was thought to be abandoned. The client confirmed that modern surface drainage from the ruined house and its access road all discharged to soakaways far upstream of the embankment, and the weather had been dry prior to and during the works. The intensity and duration of the flows also suggested an active discharge of some kind to the system.

Sonding was used to locate a series of Victorian-era masonry manholes parallel to the right-hand side of the reservoir, several of which were fully buried and exposed by the Contractor but found to be in otherwise good condition within. Clean water with drainage dye was introduced upstream and observed to enter the void from the same broken pipe. The position of the manholes was in agreement with the location of an historical sewer shown in archival records (Figure 7). The sewer trace indicated that private properties upstream and beyond the client's ownership were discharging domestic sewage treatment plant to the old network. The client instructed the manholes upstream of the embankment were to be sealed with mechanical bungs, and pumping arrangements were made to deal with inflows until a long-term solution could be determined.

The broken end of the PVC pipe, now exposed in the void by 'washing' of the outflow, was cut square and bunged with 300mm of mortar. The pipe opening below the spillway invert was also bunged in the same manner. The QCE advised that upon finding a solution to the active sewer, the remnant pipe within the embankment should be grouted or removed as pipes across the embankment would remain a reservoir safety risk.



Figure 6. Clay pipe opening below the spillway, within the void. The void extended over 2.5m upstream, to the left of the image. The broken, active PVC pipe was obscured and buried within the sunken fill and not observed until it started running during the repair works.

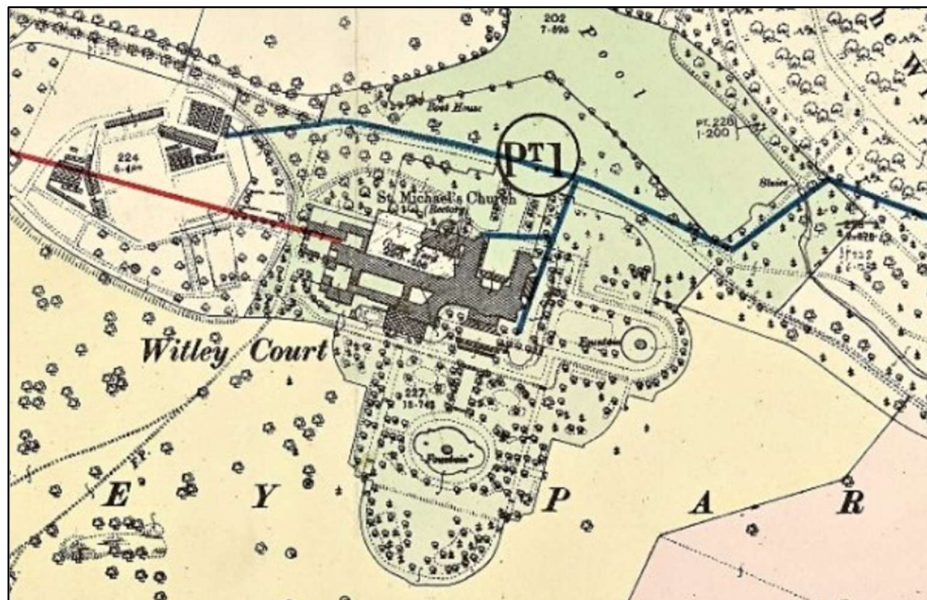


Figure 7. Extract from a plan of Witley Court Estate, found in archival records relating to the auction of the estate in 1939 following the fire. The supply main is shown in red, and the sewer in blue. Several inflow points were identified at the upstream extent of the sewer trace, while all connections from Witley Court itself are redundant.

Witley Court: Incident Response and Interim Repair

Following the temporary stoppage to the sewer pipe, drilling and grouting works resumed on the crest. No further positive connections to the void were made through drilling, and following reinstatement of the drill holes the remaining void was backfilled from the surface by tremie pipe. Grout was observed to be flowing out of the void and into the downstream embankment, with some grout migrating through the blockstone cascade to emerge below the spillway slab. Additional caulking was undertaken to contain outflowing grout, and the grout mix was thickened. Ultimately, after injection of some 3000 litres of grout (3m³), the grout levels in the void stabilised at the surface with no further seepages observed. Satisfied that the void was fully backfilled, all drill and trench locations were made good before demobilising.

DISCUSSION

The discovery of the active sewer pipe prompted re-evaluation of the erosion mechanism. Although the voided area had no doubt been compromised by long term leakage and erosion, the size of the void and the speed at which it developed (on reflection it was noted to have significantly increased in size between October 2024 and March 2025) suggest that direct washout from concentrated flows had contributed to its formation and the damage to the embankment face below.

Piecing together available evidence, it appears that the historical pipe repair referenced in the 1989 S10 was undertaken by a private landowner, and the PVC pipe section was sleeved upstream and downstream into the historical clay pipe. While the area was still affected by seepage, the loose backfill surrounding the pipe reinstatement was eroded and the repaired pipe lost support. At some later point, possibly where vertical loads were increased by the spillway bridge and crest raising works, the pipe fractured at the connection between the solid spillway structure and the embankment and direct washout started, resulting in a step change in the rate of erosion until the surface collapsed and severed the pipe completely.

The proposed reduction in the statutory raised volume to 10,000m³ for reservoirs in England is likely to introduce many historic reservoirs to regulatory controls. These reservoir structures will often have undergone many changes over time which may not be well documented, particularly where multiple changes of ownership have occurred. This incident highlights the potential risks associated with buried water pipes within dam embankments and the need to maintain accurate records. In this case, factors beyond the undertaker's knowledge or control and originating far upstream had directly contributed to a serious reservoir safety incident.



The British Dam Society

Emergency Response at an Historic Reservoir in North Wales: Emergency Drawdown and Discontinuance

E C ARCHIBALD, Stillwater Associates Ltd

J P HOLLAND, Stillwater Associates Ltd

SYNOPSIS

Llyn-du-Bach reservoir in North Wales is situated in a remote location, approximately 1.2km from the nearest road, within a UNESCO World Heritage Site. The main dam comprised masonry outer twin dry-stone walls with a compacted peat core. The reservoir was designated as High-Risk in 2022, and a Section 10 Inspection was completed later that year. The All Reservoirs Panel Engineer (ARPE) made recommendations as to measures in the interests of safety, to either carry out substantial reservoir safety improvement works or to discontinue the reservoir.

Routine monitoring identified substantial deterioration in 2025. The peat core was found to be absent from much of the structure. The dam was considered to present an imminent risk of failure, with an associated risk of flooding and potential loss of life in Blaenau Ffestiniog. The ARPE and Natural Resources Wales (NRW) agreed that the reservoir must be drawn down as soon as safely practicable. Works to achieve this were completed in December 2025 and the reservoir was subsequently permanently discontinued. This paper presents the adopted approach and discusses the challenges associated with the site's remote location, heritage designation and environmental value, and how these were overcome. It further highlights key lessons for similar projects and for the wider UK dams and reservoirs sector.

LLYN-DU-BACH RESERVOIR

Setting the Scene

Under provisions made in the Flood and Water Management Act 2010 the capacity threshold for reservoirs in Wales was lowered to include 'small raised reservoirs' with a stored volume between 10,000m³ and 25,000m³. The process followed for identifying these reservoirs also resulted in the discovery of a number of large raised reservoirs that had not previously been registered. One such reservoir was Llyn-du-Bach in North Wales, with a capacity of 54,660m³, of which 40,710m³ was assessed as being above the level of adjacent natural ground. Accordingly, Llyn-du-Bach was added to

Emergency Response at an Historic Reservoir in North Wales: Emergency Drawdown and Discontinuance

the register of large raised reservoirs, and was designated as a High Risk reservoir in June 2022.

Located in the Eryri (Snowdonia) region, the reservoir was constructed pre-1888 to support the local slate mining industry. The reservoir was first created with a low dam formed across the outlet of a natural lake, raising the water level by approximately 3m. Mapping of the area shows that the enlarged lake was further extended at some point between 1914 and 1949 with the construction of a new dam across the watercourse, some 200m downstream of the original dam. Although there are no formal ecological designations associated with the site, the site is located within the UNESCO World Heritage designated Slate Landscape of North-West Wales. CADW is the relevant authority for these sites in Wales.



Figure 1. Llyn-du-Bach reservoir site setting

The upstream catchment and valley downstream of the reservoir are steep, with the town of Blaenau Ffestiniog just 1.3km downstream and with an elevation difference of some 250m. The terrain along the routes of two watercourses downstream of the reservoir is rugged: a mixture of heathland, rocky outcrops, loose boulders and slate waste.

Access to the site is limited to indistinct public footpaths across rough heathland, taking at least 40 minutes to walk in good conditions. The nearest public road is 1.2km straight-line distance from the site. The nearest accessible track suitable for any sort of vehicle is within a working slate quarry, 1.5km from the dam.

The overall ownership of the reservoir was not immediately apparent, with three separate land registrations listed. The farmer who owned part of the reservoir, along with a second potential owner / Undertaker, the incumbent slate quarry company, agreed to appoint an Inspecting Engineer in response

to notices served by NRW. Accordingly, the first Section 10 inspection of Llyn-du-Bach reservoir was undertaken in October 2022 and reported in April 2023.

First Inspection & Reporting under Section 10 of the Reservoirs Act 1975

The dam construction was found to be typical of those associated with similar reservoirs in North Wales, consisting of two dry stone walls encasing a central peat core, evidently formed using materials found locally to the site. The age and history of the dam of course meant that there was no information available on the internal construction or foundation treatment. At the time of the Section 10 inspection the dam wall, including the peat core, appeared to be largely intact with no signs of significant movement, loose stones or loss of material. There was, however, minor leakage through the lower part of the 3m high dam.



Figure 2. Dam crest viewed from the left abutment, taken during the 2022 Section 10 Inspection

The reservoir included a narrow and shallow overflow channel cut into rock through the right bank abutment, offering low capacity and inadequate freeboard. Although there was visible evidence of a high level outlet which would once have allowed water to be released to a leat on the left abutment, this facility was no longer operational and apparently blocked.

Emergency Response at an Historic Reservoir in North Wales: Emergency Drawdown and Discontinuance

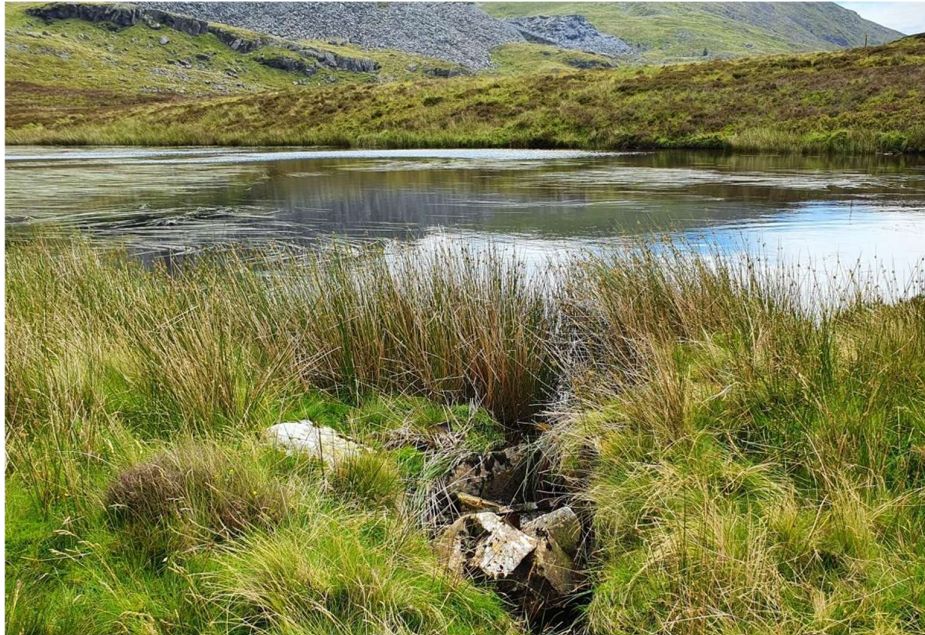


Figure 3. Overflow channel entrance obscured by vegetation growth at the time of the 2022 Section 10 Inspection

In view of the significant consequences downstream, based on mapping and data provided by NRW, and assessed as part of the Section 10 reporting, the reservoir was determined to be Category A, where a breach of the dam could endanger lives in a community.

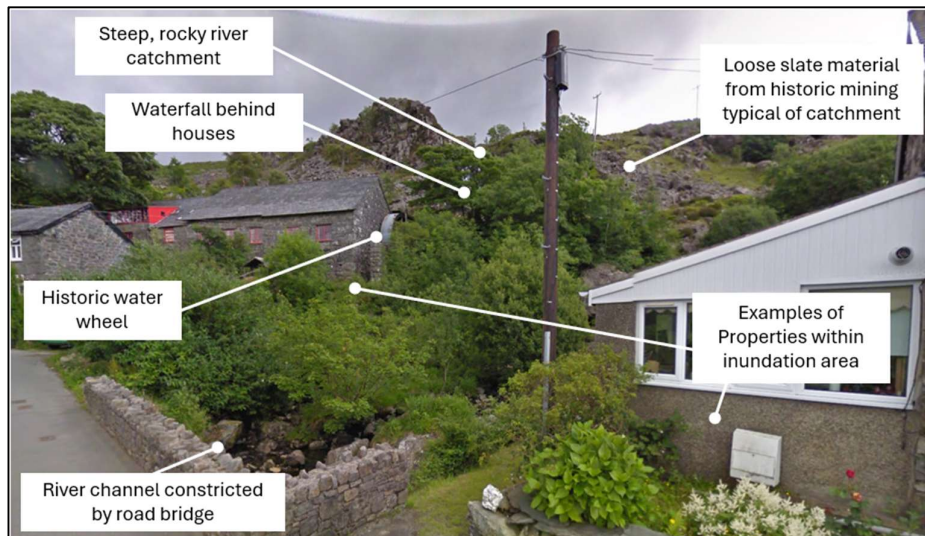


Figure 4. Annotated image of the downstream watercourse at Blaenau Ffestiniog (image courtesy of Google Maps)

Archibald and Holland

A visit to the relevant downstream area on the south side of the town revealed a number of residential properties immediately adjacent to the main watercourse from the reservoir, the closest of these being beneath a steep drop with a 20m high waterfall immediately behind houses. It was apparent that a dam breach, with an uncontrolled release of water, could result in inundation of properties from above, with the cascading water likely to be transporting boulders and loose slate pieces. It was evident that a failure of the dam had the potential for catastrophic consequences in this part of the town.

The Section 10 report included the following MIOS:

'In view of the condition of this reservoir and the risk posed to people and property I recommend that:

- 1. A decision is made in discussion with an All Reservoirs Panel Engineer to either undertake improvement works or to discontinue the reservoir followed by implementing the necessary works as follows:*
 - 1a. Works undertaken to improve the safety of the dam to satisfy current reservoir safety standards, expected to include, but not limited to:*
 - Replacing and/or improving and protecting the condition of the watertight element to address current leakage and to reduce the likelihood of future leakage;*
 - Raising the level of the dam and/or modifying the overflow to provide the required flood freeboard in accordance with current guidance.*
 - The installation of a permanent low-level drawdown facility. The available drawdown rate of the installed facility should be in accordance with current guidance and should be not less than 300mm per day.*

or;

- 1b. Works are undertaken to discontinue the reservoir in accordance with Section 13 of the Act by sufficiently removing [the dams] to ensure water can no longer be impounded behind either of these structures with a total volume not exceeding 10,000m³ above adjoining natural ground level.*

The report gave a completion date for recommendation 1 within two years of the date of the report and for either recommendation 1a or 1b within five years of the date of the report.

Emergency Response at an Historic Reservoir in North Wales: Emergency Drawdown and Discontinuance

Reservoir Supervision and Incident Reporting

The Undertakers duly appointed a Supervising Engineer (SE) and prepared the requisite reservoir documentation for a high-risk reservoir. Annual visits were made by the SE, with the production of annual statements. The SE was also employed by the Undertaker to carry out monthly surveillance visits.

During the Supervising Engineer's visit in September 2025 it was discovered that the central peat core, intended to form the watertight element for the dam, was almost entirely absent, leaving a large void between the two stone walls. The ARPE was requested to visit the site as soon as possible to advise on the required course of action. Upon investigation it became clear that the central peat core had in fact eroded and been washed out over an extended period of time with the upper area bridging the gap. This upper part was most likely reinforced, protected, and held in place by vegetation until weathering or overspilling of the dam had sufficiently weakened the upper part of the core for it to fall into the void beneath. Leakage through the dam had significantly increased, flowing mainly from the base of the dam.



Figure 5. Dam crest viewed from the right abutment in 2025, with missing peat core



Figure 6. Dam wall viewed from downstream, showing area of leakage at the dam toe

With concern that the leakage at or just above foundation level could wash out material supporting the dam walls, it was clear that measures needed to be taken to make the reservoir safe. The usual course of action in these circumstances would be to release water from the reservoir to lower the

water to a 'safe' level as a temporary measure until works are carried out to address the prevailing issue. In this case there was no installed means of releasing water from the reservoir.

An incident was declared with NRW and discussions started in relation to prompt actions that could be taken to make the reservoir safe.

RESERVOIR INCIDENT: INITIAL RESPONSE

The initial response was to initiate a reservoir drawdown in a timely manner. The installation of pumps or siphon pipes was considered. The logistics of this approach with no road access was such that an immediate response would have required airlifting of plant and materials to the site. Whilst this was considered feasible, it was agreed that the ongoing logistics of maintaining the operation of pumps or siphons once installed would present significant challenges. In discussion with NRW it was agreed that a managed dismantling of the dam was both feasible and acceptable in terms of achieving a controlled emergency lowering of the reservoir water level, ultimately resulting in discontinuance of the reservoir. Immediate measures involved surveillance visits three times a week by the Supervising Engineer, keeping the ARPE apprised of the condition of the dam and any changes in leakage, in conjunction with a full off-site response with relevant Category 1 responders on standby. A Tactical Coordination Group (TCG) was established with regular Teams calls to keep the TCG updated with any changes in the situation and progress with accessing the site to dismantle the dam.

Recognising the urgency of the situation, a lead Undertaker was agreed upon who could coordinate and support the on-site response under the guidance of the ARPE. The key aspects managed by the Undertaker were:

- Identifying and appointing an engineering contractor with appropriate experience of reservoir related work and who could respond promptly.
- Liaising with the relevant authorities and key stakeholders:
 - Natural Resources Wales
 - Gwynedd Council (the Local Planning Authority)
 - CADW
 - The second Undertaker / landowner
 - The local slate quarry owners and operators
 - Ecology specialists
 - Archaeology specialists
 - Local residents
 - ARPE

Emergency Response at an Historic Reservoir in North Wales: Emergency Drawdown and Discontinuance

- o The Tactical Coordination Group
- Providing regular and carefully formulated communications for residents within the possible inundation zone, to pre-empt the inevitable public response and with a view to the potential for an emergency evacuation.

Design details for dismantling of the dam and subsequent discontinuance were prepared under the supervision of the ARPE and in discussion with the appointed contractor, chosen for their experience in reservoir discontinuances in north Wales. The works comprised:

- Provision of temporary access to the reservoir site, constructed through a combination of local excavation, placement of granular fill and bog matting.
- Initial drawdown, achieved by clearing and excavating the existing, partially blocked spillway channel, to increase the channel capacity and lower the level of the overflow.
- Creation of a temporary stone spillway downstream of the dam using stone relocated from the downstream part of the dam wall.
- Temporary infilling of the dam core with stone relocated from the upstream part of the dam wall.
- Controlled lowering of the dam wall in increments not greater than 100mm.
- Relocation of stone from the dam downstream wall to form an apron to protect the downstream gully.
- Returning the pre-existing natural lake to its original form on completion of the discontinuance.

The details were presented to relevant stakeholders along with information on the prevailing risk to the downstream population. In all cases the stakeholders acknowledged the urgent nature of the works and committed to taking a proactive and pragmatic approach to developing appropriate mitigations to address the environmental, heritage, legal and logistical constraints associated with the scheme. The main aspects of these are discussed below.

RESERVOIR INCIDENT: RESPONSE CONSTRAINTS

Heritage and Archaeological Constraints

Upon identifying the UNESCO status of the site and on the advice of Gwynedd Council, CADW was engaged to review the proposed works and provide guidance on the necessary mitigation measures to protect the site's heritage value. CADW confirmed that the reservoir drawdown would not negatively affect the Outstanding Universal Value of the Slate Landscape of Northwest Wales World Heritage Site, but required that phased dismantling of the dam be completed in a manner compatible with World Heritage values. This included a requirement to record the dam structure by photogrammetric survey and to retain sections of the masonry in-situ, on either abutment.

Construction of the temporary access track was perhaps the more challenging aspect of the works, from a heritage perspective. This was in part due to programme constraints, being one of the first activities completed following mobilisation, but also because of the presence of historic quarry features along the access route, including slate tips, a weigh house and other buildings. The slate tips at Eastern Diffwys, in particular, have distinctive 'fingertip' shapes and CADW wanted to ensure that the profile of the tip edge was preserved.

Heritage mitigation measures were agreed through the preparation of a Written Scheme of Investigation (WSI). This was delivered in stages to allow site mobilisation and access works to commence in advance of the WSI being agreed for the dam itself. The WSI required auger sampling of peat in areas where bog mats were proposed for the access track, where there was considered to be palaeo-environmental potential. Following an archaeological inspection, minor route adjustments were made, resulting in only a small portion of the access track being sited on peat.

Environmental Constraints

From an environmental point of view, one of the main challenges was managing silt, given the highly erodible nature of the catchment. Measures to protect against the washing of silt into the downstream watercourse were employed during the works and post-completion. Measures during the works comprised coir rolls and hay bales placed in the downstream channel. To provide long-term protection, it was proposed to relocate silt within the basin in stages corresponding to the reduction of the dam crest. However, it was necessary to adapt the long term measures following the lowering of the reservoir, as the silt depth was significantly more than indicated by the bathymetric survey. The large quantities of silt could not be safely deposited

Emergency Response at an Historic Reservoir in North Wales: Emergency Drawdown and Discontinuance

away from the reservoir, without a significant environmental impact. Alternatively, the silt was retained by further hay bales and coir rolls along a natural channel in the reservoir basin. These measures were discussed and agreed on-site with the contractor, prior to buy-in from the project ecologist. Post-completion, sign posts were erected around the reservoir, warning of deep silt.

Legal Constraints

The works completed at Llyn-du-Bach would ordinarily require Planning Permission, Ordinary Watercourse Consent and/or an Impoundment Licence. However, due to the urgent nature of the works, NRW and Gwynedd Council agreed that the formal licence application processes would not need to be followed. This was contingent on consultation and ongoing dialogue with NRW and CADW, and on the agreement and implementation of appropriate heritage and ecological mitigations, as previously discussed.

Logistical Constraints

The timing of the works added another layer of complexity to project delivery. Construction commenced in November 2025, and it is fair to say that the weather conditions, whilst not unusual, were sub-optimal. High winds, heavy rain and snowfall all delayed the works.

Public perception of the project also proved challenging in two main ways. Word about the project had spread, and some local residents expressed concerns about public safety. In response, river level monitoring was installed on the downstream watercourse, giving real time data which might warn of unusually raised levels, and to provide reassurance. There were also concerns from members of the community about loss of the reservoir and its amenity value, particularly in the context of other recent reservoir discontinuances in the region. Community engagement for the project focused on clearly communicating the risk associated with the reservoir and the public safety justification for the works.

DRAWDOWN AND DISCONTINUANCE

The preliminary drawdown, achieved by increasing the overflow channel capacity, was completed prior to the Christmas shutdown, significantly reducing the risk over this period. This proved to be more successful than had been anticipated, with highly weathered rock encountered for up to a metre below the original channel bed. Whilst this significantly reduced the normal level of water in the reservoir, the channel width still posed a significant constraint on overflow capacity to accommodate higher flows.

Archibald and Holland

Several mitigations were put in place for the Christmas shutdown period. Monitoring visits continued whilst the site was shut, members of the contractors' team were placed on standby and bulk bagged stone was placed at the site compound for airlifting to the dam site if the condition of the dam deteriorated.

Upon re-mobilising in the new year, steady progress was made with the works carefully supervised and controlled. An Environmental Clerk of Works was present with regular visits, along with an Archaeological Watching Brief recording the nature of the dam and its features as it was dismantled.

The controlled dismantling proved to be a successful approach, even with some higher flows through the reservoir during wetter periods. Silt within the reservoir basin was carefully managed, with silt capture arrangements in place at the location of the dam and downstream. Silt retention measures were installed as the reservoir level was drawn down with the reservoir basin becoming accessible.



Figure 7. Completed drawdown and dam removal with silt mitigation and silt retention measures being added

KEY OUTCOMES

There are numerous dams in North Wales of a similar construction to Llyn-du-Bach and as reservoir engineers, we must ask ourselves, do we properly understand their condition, even following visual inspections? If the reservoir is 'High Risk' should more detailed investigations be carried out to check their condition, especially where construction records are limited or non-existent? It is understood that the Reservoir Safety Reform Programme is seeking to address this question by developing a Reservoir Safety Case that is tied to the proposed new classification system. It is currently unclear whether the Reservoir Safety Case will go as far as requiring on-site investigations and / or detailed assessments for existing reservoirs, and it is expected that the emphasis will be on achieving a risk-based and

Emergency Response at an Historic Reservoir in North Wales: Emergency Drawdown and Discontinuance

proportionate response. Nevertheless, there is a risk that any such change may result in onerous requirements for the owners of small raised reservoirs. As a long term strategy, could a regional review of 'High Risk' reservoirs be considered, particularly for those in private ownership, with a view to discontinuing reservoirs, or reducing the stored volume to a safe level, where the reservoir no longer serves a useful purpose?

For remote reservoir sites with inadequate access for bringing plant and materials to site at short notice, installed drawdown facilities should be reviewed with reference to current guidance. Where required, appropriate facilities should be installed, to avoid reliance on mobile / temporary measures, or improvement works carried out to the access, possibly as a measure in the interests of safety to allow for emergency access with temporary pumping equipment.

Complications can arise where there is more than one owner. This project benefitted from nominating a lead Undertaker who was able and ready to respond promptly, and this might be an advantage in other similar situations. In this instance, it was fortunate that one of the owners had the resources and capability to address the various strands required to respond to the incident in a timely manner. The outcome may have been different if the reservoir had been owned by one of the many Undertakers who are not in this position.

It is recognised that under normal circumstances the relevant permissions and licences should be obtained before commencing any significant works. However, it could perhaps be questioned whether there is a need for a Planning Application, an Impoundment Licence and an Ordinary Watercourse Consent on reservoir discontinuance projects, where just one of these could address all relevant aspects of a scheme. Or, going further, whether these might be combined into a single streamlined process.

This project has demonstrated that, where there is an immediate risk or emergency situation, it is possible to implement reasonable mitigation measures, with an appropriate level of stakeholder engagement and input, that achieve acceptable environmental and heritage outcomes. There may be a benefit to adopting a similar collaborative and proactive approach more widely where the discontinuance of an historic reservoir is deemed to be the most appropriate response to address the prevailing risk to life.



The British Dam Society

Learning from success and failure – embankment dams founded on high-plasticity over-consolidated clays

P TAYLOR, AtkinsRéalis
A CAMPBELL, AtkinsRéalis
S QI, AtkinsRéalis
A CÁCERES-TORRES, AtkinsRéalis
N GREEN, AtkinsRéalis

SYNOPSIS

The UK is entering a renewed period of reservoir construction requiring new embankment dams founded on high-plasticity over-consolidated clays that exhibit strain-softening behaviour. Progressive failure must therefore be allowed for in the mobilised strengths adopted for limit equilibrium analysis. The selection of appropriate effective strengths has a controlling influence on slope geometry.

This paper presents a framework, built from existing literature, for selecting average mobilised strengths at failure, for use in limit equilibrium stability analyses. The strengths are aligned with the recommended factors of safety given by U.S. Department of the Interior (2011). Used with best-estimate unit weight and pore water pressure, this approach provides a rational basis for slope geometry, achieving an appropriate balance between safety, cost and programme.

The approach initially derives bulk strength that accounts for the effect of weathering and minor discontinuities before applying further reductions to account for progressive failure. Major shear surfaces where present require separate explicit consideration.

Bulk strength may be estimated using larger-diameter triaxial testing or reduction of higher intact strengths from smaller triaxial or shear box testing. Evidence indicates mobilised strength at failure is approximately midway between bulk and residual values.

Back-analysis of the 1937 Abberton Dam failure gives mobilised strength values of $\phi' = 16^\circ$ and $c' = 0$ kPa, consistent with the $\phi' = 16$ to 17° and $c' = 0$ kPa range predicted by the framework, supporting its application to contemporary UK schemes.

Learning from success and failure – embankment dams founded on high-plasticity over-consolidated clays

INTRODUCTION

The selection of effective strength parameters for high-plasticity over-consolidated clays beneath embankment dams has a controlling influence on slope geometry, safety and cost. These materials exhibit strain-softening behaviour and are prone to progressive failure, but limit equilibrium analyses cannot capture such behaviour explicitly. Rather, average mobilised strength at failure is required.

Establishing this mobilised strength is central to defining slope geometry and meeting required factors of safety, such as those recommended by U.S. Department of the Interior (2011). Used with best-estimate unit weight and pore water pressure, this provides a rational and transparent basis for design, achieving an appropriate balance between safety, cost and programme.

This paper addresses the derivation of mobilised strength for limit equilibrium analysis, using sound engineering judgement informed by precedent, field observations, testing and analysis. It considers how bulk strength is reduced by weathering and minor discontinuities, including fissures and non-persistent shear surfaces. Separately, where major persistent and unfavourably oriented shear surfaces are present, explicit modelling as shear zones mobilising field residual strength is recommended.

A framework is developed for deriving bulk strength and reducing it to mobilised strength to represent conditions at failure for limit equilibrium analysis. The resulting strength should be verified, where appropriate, using strain-softening numerical analysis.

CONTEXT – UK DAMS AND CONTEMPORARY NEED

Learning from precedent

The current phase of UK reservoir construction is largely founded on high-plasticity over-consolidated Jurassic, Cretaceous and Paleogene clays and mudstones. Figure 1 shows these current schemes alongside a dataset of 21 comparable UK embankment dams founded on similar materials, compiled to inform parameter selection from observed performance.

Observed performance of UK dams

Published sources and as-built records for the 21 dams were reviewed to compare embankment geometry with observed performance (Figure 2). The dams with extremely low crest to toe slope angles have toe berms to improve stability and are much steeper towards their crests.

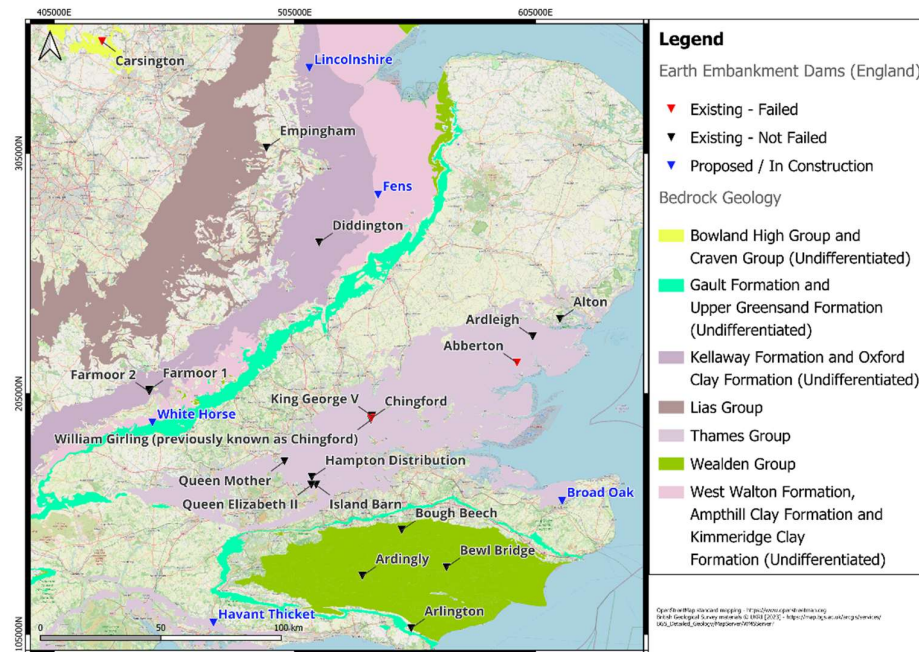


Figure 1. UK embankment dams on high-plasticity over-consolidated clays

Three dams underwent deep-seated failures through their foundation clays towards the end of construction: Abberton, Carsington and Chingford. No similar deep-seated failures have been identified during first filling or long-term operation, and none of the failed slopes was flatter than 1V:4H (14°).

Slope gradients show wide scatter and only limited correlation with embankment height. This likely reflects foundation conditions, adopted strengths, pore water pressures, construction rates, risk tolerance and, in some cases, design pre-dating modern soil mechanics. Where no toe berms were used slopes of about 1V:5H to 1V:7H (8° to 11°) are broadly associated with satisfactory performance and provide a useful reference range for contemporary schemes on comparable materials.

There are outliers. Empingham Dam, for example, is founded on Upper Lias Clay (Lias Group) heavily sheared by valley bulging, yet final design checks gave factors of safety greater than 1.6 (Bridle et al., 1985).

STRENGTH BEHAVIOUR OF HIGH-PLASTICITY OVER-CONSOLIDATED CLAYS

The design of embankment dams founded on clay strata typically relies on effective stress stability analyses, using the Mohr–Coulomb strength envelope, $\tau' = c' + \sigma'_n \tan \phi'$, where τ' is strength, c' cohesion, σ'_n normal stress and ϕ' the friction angle, all in effective stress terms.

Learning from success and failure – embankment dams founded on high-plasticity over-consolidated clays

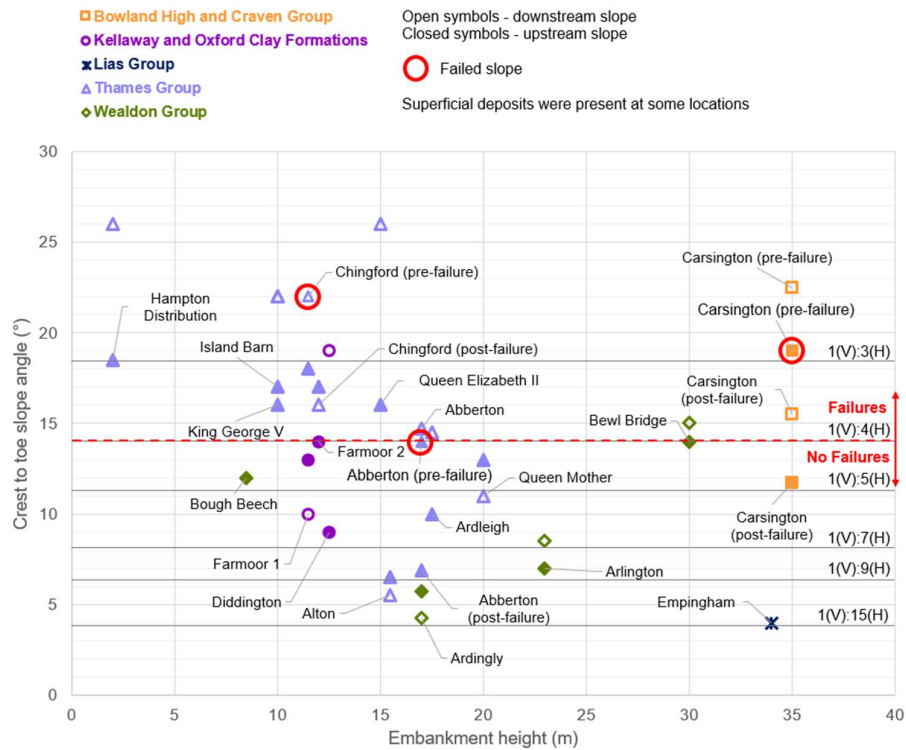


Figure 2. Existing UK embankment dams: height-slope relationship

This paper only considers the selection of constant c' and ϕ' values, which exert a principal control on embankment slope geometry. In reality, the Mohr–Coulomb strength envelope is moderately non-linear, so for any given analysis c' and ϕ' should be selected to represent strength at appropriate normal effective stresses.

The case studies report stress levels and pore water pressures, so the findings on strength parameter selection are robust. Pore water pressure is equally important because it controls normal effective stress.

The idealised shear strength-displacement behaviour of intact high-plasticity over-consolidated clays is presented in Figure 3. Peak strength is mobilised at small displacement, followed by strain softening to a fully softened condition, taken here as the normally consolidated drained peak. In high-plasticity clays this is approximately equal to the onset of critical state or constant-volume shearing. Continued shearing leads to progressive development of a polished shear surface, with eventual mobilisation of laboratory residual strength.

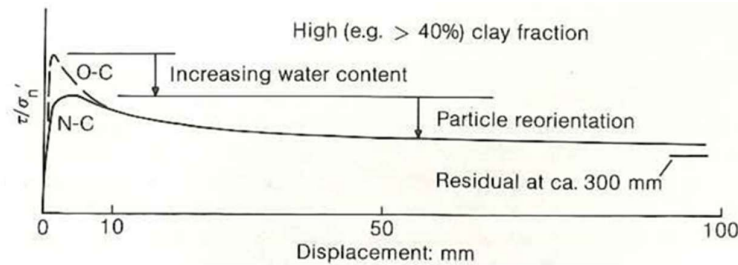


Figure 3. Shear strength–displacement curve (Skempton, 1985)

Controls on Bulk Strength

In situ bulk strength is reduced from intact peak values by weathering and minor discontinuities, both of which co-exist in high-plasticity over-consolidated clays.

Weathering

Weathering reduces the strength of high-plasticity clays through fabric degradation and loss of apparent over-consolidation. This reduces effective cohesion, while the friction angle remains broadly unchanged, Chandler and Apted (1988).

This implies increasing strength with depth from weathered to unweathered clay, consistent with observations from London Clay (Thames Group) cuttings, Skempton (1977). This contrast in strength and most likely stiffness may promote strain localisation at the base of the weathered zone.

Although these observations are derived from London Clay, the underlying mechanisms are likely to be characteristic of high-plasticity over-consolidated clays more generally.

Minor discontinuities

Minor discontinuities comprise fissures, joints and discontinuous shear surfaces within the clay mass. These do not form persistent failure planes at embankment scale, but they reduce intact peak strength to bulk strength and may mobilise low shear resistance at relatively small displacements, Skempton and Petley (1967). Their effect is best represented through reduced bulk strength. In the laboratory, sample size influences the measured strength. Small shear box tests and 38mm diameter triaxial tests are less likely to contain discontinuities and therefore more likely to provide intact peak strength, whereas 100mm diameter triaxial tests are more likely to contain discontinuities and provide an approximate measure of bulk strength. This sample-size effect is widely recognised, for example in relation to undrained strength by Patel (1992).

Learning from success and failure – embankment dams founded on high-plasticity over-consolidated clays

Major shear surfaces

Major shear surfaces are laterally persistent and can form preferential failure planes. They are best represented explicitly in stability analyses as shear zones with field residual strength and observed field orientation.

Major shears may arise, for example, from tectonic shearing, valley bulging, periglacial shearing, and relict landsliding. Their engineering significance depends on persistence, three-dimensional orientation and strength. Where such shears are suspected, ground investigation should seek to establish their origin, extent and orientation so that the hazard they present can be managed explicitly in design.

FRAMEWORK

This paper presents a framework, built from existing literature, to assist selection of mobilised strength for limit equilibrium slope stability analysis of embankment dams founded on high-plasticity over-consolidated clays. It derives bulk strength by accounting for weathering and minor discontinuities within the clay mass, then reduces this to mobilised strength to allow for progressive failure.

The geotechnical triangle (Burland, 1987) provides a useful context for the framework (Figure 4). In the triangle, robust parameter selection depends on balancing ground characterisation, observed behaviour and modelling, underpinned by precedent, empiricism and well-winnowed experience.

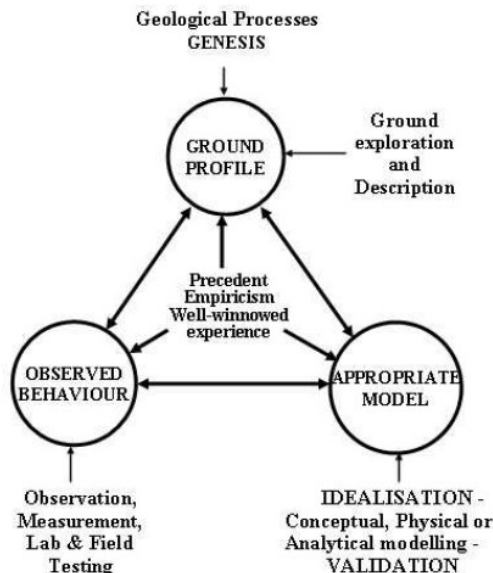


Figure 4. The geotechnical triangle

The framework draws these strands together to support strength parameter selection based on sound engineering judgement.

Phase 1 – Literature review

Precedent and experience provide an essential starting point for parameter selection but are often underused in modern practice. Phase 1 should review case histories to establish the characteristic behaviour and typical parameter values of the expected founding strata.

Phase 2 – Estimating bulk strength

The starting point for Phase 2 should be carefully planned ground investigation, including observation trial pits to forensically record the strata and discontinuities, high quality sampling and strength testing. The interpreted results should then be checked against the findings of Phase 1.

Fully-softened strength

Fully-softened strength is associated with shallow slope failures, which develop as seasonal wetting and drying progressively reduce strength. This degree of softening is not credible in stiff clays beneath embankment dams. Chandler (1984) noted that agreement between back-analysed field strengths and fully-softened laboratory strengths is “surprising and perhaps fortuitous”, reflecting the difficulty of attaining a fully-softened state at depth within engineering timescales.

In practice, however, fully-softened strength sits in the middle of the geotechnical triangle, linking precedent, empiricism and well-winnowed experience with observed behaviour and modelling. There is strong evidence that fully-softened strength approximates bulk strength, with mobilised strength at failure lying between fully-softened and laboratory residual strength. It is therefore a useful reference for parameter selection. It can be estimated using published values, correlations such as Stark and Eid (1997), and, where practicable, testing on reconstituted clay samples.

Estimating bulk strength

Bulk strength may be estimated from triaxial testing of large-diameter samples or, less commonly, from plate bearing tests taken to failure in the field. The reliability of the derived bulk strength depends on how well the minor discontinuities within the tested volume reflect the soil mass.

Peak intact strength can be estimated reliably from laboratory testing of small intact samples. Combined with suitable measurements or estimates of discontinuity strength, this allows the two methods proposed by Skempton and Vaughan (1993), based on the Carsington Dam failure, to be used to

Learning from success and failure – embankment dams founded on high-plasticity over-consolidated clays

estimate bulk strength. Where discontinuous shear surfaces are approximately aligned with likely failure mechanisms, bulk strength may be estimated by area-weighting intact and shear-zone strengths, with shear-zone strength taken as laboratory residual increased to account for surface undulation (the 'Yellow Clay' method). Where minor discontinuities are more random or less favourably oriented, approaches after Vaughan (1985) and Potts et al. (1990) may be used (the 'Clay Core' method).

Direct measurement and estimates based on the Carsington methods can both be used to investigate bulk strength. Agreement between them, if achieved, increases confidence.

Phase 3 – Estimating mobilised strength for limit equilibrium analysis

The reduction from bulk strength to average mobilised strength at failure, to account for progressive failure, can be represented using a residual factor, $R = (\tau_{\text{bulk}} - \tau_{\text{mob}}) / (\tau_{\text{bulk}} - \tau_{\text{res}})$, Skempton (1964). Back-analysis of the Carsington Dam failure gave $R = 0.52$ for the Yellow Clay in the foundation, Skempton and Vaughan (1993).

Stark and Eid (1997), based on 14 case histories including cut slopes and embankment dams, showed that high-liquid-limit clays mobilised strength at failure approximately midway between fully softened and laboratory residual values, consistent with $R = 0.50$ (Figure 5).

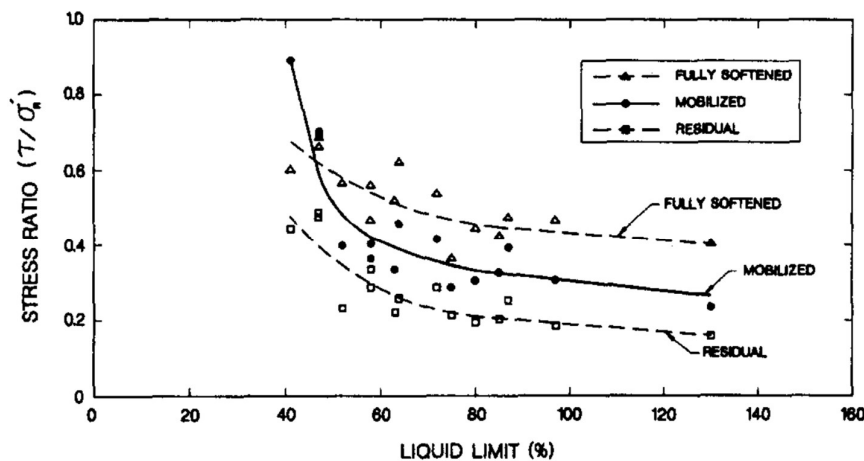


Figure 5. Mobilised strength at failure (Stark and Eid, 1997)

Potts et al. (1997) reported residual factors with an average of $R = 0.56$ (range 0.39 to 0.69) from finite element analyses of cuttings in Weathered London Clay. Alonso and Gens (1999) reached similar conclusions in their analysis of the Aznalcóllar tailings dam failure.

Taken together, these studies support adoption of $R = 0.50$ as a pragmatic allowance for progressive failure in limit equilibrium analyses of high-plasticity over-consolidated clays.

USING THE FRAMEWORK

The three phases of the framework are not intended to be applied as a rigid sequence. Rather, they provide complementary lines of evidence, aligned with the geotechnical triangle, that should be selected and combined according to the ground conditions and design question under consideration.

APPLICATION TO ABBERTON DAM

As the failure and strain-softening numerical analysis of Abberton Dam were reported in detail by Kovacevic et al. (2013), this case study has been selected to test and demonstrate use of the framework. The earth embankment dam failed during construction in 1937 at a height of 13m with slopes of 1V:4H.

Phase 1 – London Clay literature review

Skempton (1977) reported mobilised strength of $\phi' = 20^\circ$ and $c' = 1$ kPa across a range of London Clay sites, which provide a credible upper estimate of mobilised strength. Fully-softened strength, taken as $\phi' = 20^\circ$ and $c' = 0$ kPa, provides a reference for bulk strength.

Abberton-specific intact peak strengths reported by Kovacevic et al. (2013), $\phi' = 24^\circ$ and $c' = 8$ kPa, together with joint strengths from Wraysbury reported by Skempton and Petley (1967), $\phi' = 16^\circ$ and $c' = 0$ kPa, are used to derive bulk strength using the randomly oriented discontinuity approach.

Laboratory residual strength is taken as $\phi' = 12^\circ$ and $c' = 0$ kPa from testing across a range of sites reported by Skempton (1985). A lower bound for mobilisation is therefore taken as $\phi' = 14^\circ$ and $c' = 0$ kPa, reflecting field residual strength and allowance for surface undulation, after Nowak and Gilbert (2015) and Dounias et al. (1988).

Phase 2 – Estimating bulk strength

There is no evidence at Abberton Dam for major persistent shear surfaces, or for discontinuities sufficiently continuous and oriented to control failure. Bulk strength is therefore derived using the 'clay core' method, with peak intact strength from Kovacevic et al. (2013) and minor discontinuity strength from Skempton and Petley (1967). This gives $\phi' = 22^\circ$ and $c' = 2$ kPa, slightly above fully softened strength and consistent with minor-discontinuity-controlled bulk strength.

Learning from success and failure – embankment dams founded on high-plasticity over-consolidated clays

Phase 3 – Estimating mobilised strength

Bulk strength must be reduced to account for progressive failure. Using the parameters established in Phases 1 and 2, and adopting a residual factor of $R = 0.50$, the following mobilised strengths are obtained:

- Fully-softened to laboratory residual midpoint: $\phi' = 16^\circ$ and $c' = 0$ kPa
- Randomly oriented discontinuity approach: $\phi' = 17^\circ$ and $c' = 1$ kPa

Prediction

Mobilised strength is therefore estimated to be $\phi' = 16^\circ$ to 17° , with effective cohesion (c') close to zero. The wider range of $\phi' = 14^\circ$ to 20° provides context for the likely bounds of mobilised strength, but the most credible values for Abberton Dam are $\phi' = 16^\circ$ to 17° and $c' = 0$ kPa.

Back-analysis of the Abberton Dam failure

A limit equilibrium back-analysis was undertaken to validate the mobilised strengths predicted using the framework, based on the dam failure.

The back-analysis (Figure 6) adopts the geometry, material properties, pore water pressures and failure mechanism reported by Kovacevic et al. (2013), who attributed failure to progressive failure within the London Clay foundation under high construction pore water pressures, represented by $\bar{B} = 0.8$. This value was cross-checked using a finite-difference approximation based on the reported filling rate of 13m over 32 weeks and a credible range of coefficients of consolidation. Good agreement with the finite element model pressures was obtained.

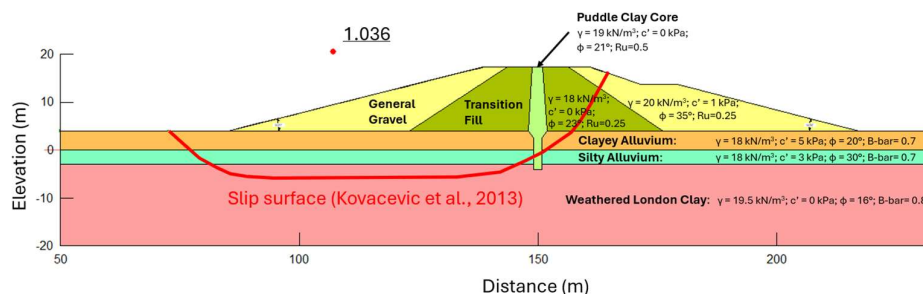


Figure 6. Limit equilibrium back-analysis of the Abberton Dam failure

Reducing ϕ' until the factor of safety is approximately unity gives a mobilised friction angle of approximately 16° with no effective cohesion at failure.

CONCLUSIONS

The selection of effective strengths exerts a principal control on embankment dam design founded on high-plasticity over-consolidated clays.

This paper presents a framework in which bulk strength is derived from testing, supported by estimates using intact strength, minor discontinuity strength and fully-softened reference values, then reduced to average mobilised strength at failure to allow for progressive failure. Applied to the 1937 Abberton Dam failure, the framework predicts $\phi' = 16^\circ$ to 17° with $c' = 0$ kPa, consistent with back-analysis giving $\phi' = 16^\circ$ and $c' = 0$ kPa.

Major shear surfaces, where present, require separate consideration and explicit modelling.

The framework provides a practical basis for selecting average mobilised strengths at failure for contemporary UK embankment dam schemes on high-plasticity over-consolidated clays. It supports proportionate design, grounded in precedent, case history and analysis, while retaining the site-specific engineering judgement required by ground uncertainty.

REFERENCES

- Alonso E E and Gens A (1999). The Aznalcóllar dam failure, Part 2: Stress–deformation analysis. *Géotechnique*, **49**(5), pp. 615–640.
- Bridle R C, Vaughan P R and Jones H N (1985). Empingham Dam – design, construction and performance. *Proceedings of the Institution of Civil Engineers, London, Part 1*, April, 78, pp. 247–289.
- Burland J B (1987). The teaching of soil mechanics – a personal view. *Proceedings of the 9th European Conference on Soil Mechanics and Foundation Engineering*, Dublin, 3, pp. 1427–1447.
- Chandler R J (1984). Recent European experience of landslides in over-consolidated clays and soft rocks. In: *Skempton A.W. and Petley D.J. (eds.), Proceedings of the 4th International Symposium on Landslides*, Toronto, 1, pp. 111–126.
- Chandler R J and Apted J P (1988). The effect of weathering on the strength of London Clay. *Quarterly Journal of Engineering Geology*, **21**. London, pp. 59–68.
- Dounias GT, Potts D M and Vaughan P R (1988). The shear strength of soils containing undulating shear zones: a numerical study. *Canadian Geotechnical Journal*, **25**, pp. 550–558.

Learning from success and failure – embankment dams founded on high-plasticity over-consolidated clays

- Kovacevic N, Hight D W, Potts D M and Carter I C (2013). Finite-element analysis of the failure and reconstruction of the main dam embankment at Abberton Reservoir, Essex, UK. *Géotechnique*, **63(9)**, pp. 753–767.
- Nowak P and Gilbert P (2015). *Earthworks: a guide, 2nd ed.*, ICE Publishing, London
- Patel D C (1992). Interpretation of results of pile tests in London Clay. In: *Piling – European practice and worldwide trends. Proceedings of a conference organized by the ICE*, **15**, London pp. 100–110.
- Potts D M, Dounias G T and Vaughan P R (1990). Finite element analysis of progressive failure of Carsington embankment. *Géotechnique*, **40(1)**, pp. 79–101.
- Potts D M, Kovacevic N and Vaughan P R (1997). Delayed collapse of cut slopes in stiff clay. *Géotechnique*, **47(5)**, pp. 953–982.
- Skempton A W (1964). Long term stability of clay slopes. *Géotechnique*, **14(2)**, pp. 77–101.
- Skempton A W (1985). Residual strength of clays in landslides, folded strata and the laboratory. *Géotechnique*, **35(1)**, pp. 3–18.
- Skempton A W (1977). Slope stability of cuttings in Brown London Clay. *Proceedings of the 9th International Conference on Soil Mechanics and Foundation Engineering (ICSMFE)*, **3**, Tokyo, pp. 261–270.
- Skempton A W and Petley D J (1967). The strength along structural discontinuities in stiff clays. *Proceedings of the Geotechnical Conference*, **2**, Oslo, pp. 29–46.
- Skempton A W and Vaughan P R (1993). The failure of Carsington Dam. *Géotechnique*, **43(1)**, pp. 151–173.
- Stark T D and Eid H T (1997). Slope stability analyses in stiff fissured clays. *Journal of Geotechnical and Geoenvironmental Engineering*, **123(4)**, pp. 335–343.
- U.S. Department of the Interior (2011). *Design Standards No. 13: Embankment dams, Chapter 1: General design standards. DS-13, Phase 4 (Final)*, Bureau of Reclamation, Denver, CO, USA
- Vaughan P R (1985). Discussion on questions raised by the Carsington Dam slide. Influence of random discontinuities on shear strength, and lateral load transfer. *Proceedings of the 11th International Conference on Soil Mechanics*, **5**, San Francisco, pp. 2817–2818.



The British Dam Society

Overview of the Project Pollaphuca Erosion Remediation Works detailing reservoir erosion remediation works and associated lessons learned

P POWER, ESB
C SMITH, ESB
O OMISAKIN, ESB

SYNOPSIS

Pollaphuca Reservoir, near Blessington, County Wicklow, is part of the River Liffey Hydroelectric Scheme and is the largest artificial reservoir in Ireland by capacity and surface area. The reservoir perimeter has experienced persistent erosion and local slope instability over many years, leading to ground loss and impacts on adjacent properties. A significant flood event in November 2009 contributed to exceptionally high reservoir levels and accelerated shoreline deterioration, prompting subsequent inspections and periodic surveys around the reservoir margin.

This paper describes the planning, investigation, optioneering, design and construction of erosion remediation works delivered at Burgage More during 2024 and 2025. The site comprises steep banks (approximately 10m to 15m height) between the reservoir shoreline and a leisure complex and public amenities including the Blessington Greenway. Remediation required management of complex constraints: fluctuating reservoir levels; difficult shoreline access; temporary works risk on marginally stable slopes; and environmental sensitivities, including the identification of tufa (petrifying) springs requiring modification of the drainage approach.

The selected solution comprised regrading of unstable upper slopes, installation of a groundwater cut-off drainage feature, and construction of a rock-armour toe berm including a toe trench along the shoreline. Delivery relied on staged excavation, survey-based movement monitoring with trigger levels, controlled exclusion zones, and a temporary access strategy integrated into permanent works. Substantial landscaping was completed to restore visual amenity and integrate the finished scheme with the surrounding recreational setting. Lessons learned are presented for future reservoir shoreline remediation projects.

INTRODUCTION

Pollaphuca Reservoir forms the upstream impoundment of the River Liffey Hydroelectric Scheme, owned and operated by ESB. Downstream of the reservoir are three Category A concrete dams and associated

Overview of the Pollaphuca Erosion Remediation Works detailing Reservoir Erosion Works and associated Lessons Learned

impoundments. The scheme is multi-purpose, supporting electricity generation, water supply, recreation, and it plays an important role in managing water upstream of Dublin.

Pollaphuca Reservoir has a surface area exceeding 20km² and a stored volume greater than 160 million m³. Persistent shoreline erosion around the reservoir perimeter has been observed over many years, with consequences including loss of land and encroachment towards adjacent properties in several locations.

A major flood event in November 2009 resulted in exceptionally high reservoir levels and was associated with further deterioration of shoreline conditions. Follow-up walkovers and boat-based inspections were undertaken subsequently. Based on these inspections, ESB identified a duty of care to manage erosion risks where reservoir operations were considered to contribute to ground loss affecting neighbouring lands.

SITE CONTEXT AND PROBLEM DEFINITION:

Burgage More is located south of Blessington, County Wicklow, adjacent to a resort complex (Figure 1). The affected shoreline comprises steep banks with evidence of localised failures, severe erosion, gully development, step features, and slumping of topsoil (Figure 2). These features indicated a deterioration trajectory that posed risk to amenities located upslope, including a public walkway.

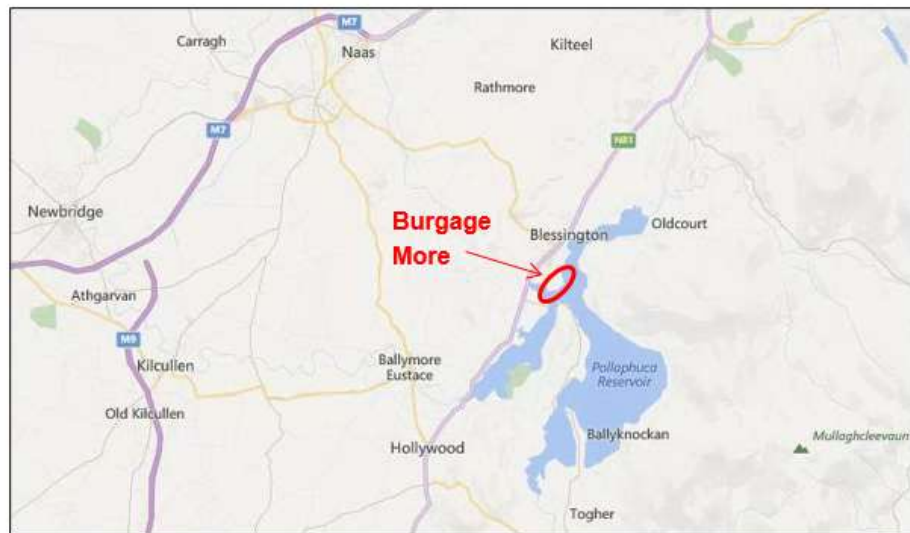


Figure 1. Pollaphuca Reservoir



Figure 2. Erosion identified at Burgage More prior to remediation works

Planning consent for the repair works was granted via a Section 5 Declaration under the Planning and Development Act (2001), confirming exempted development status for the works.

The reservoir and adjacent areas include environmental designations (including Special Protection Area designation noted for the reservoir extents/ adjacent lands), requiring structured environmental input during design development and construction planning.

SITE OVERVIEW AND RELEVANT STAKEHOLDERS

Key stakeholders included ESB Engineering and Major Projects (delivery), ESB Generation and Trading (client), the Avon Ri complex (adjacent land user), Wicklow County Council (local authority and lands used for compound/material storage), private landowners and homeowners, and environmental/ecological specialists. The Blessington Greenway, which runs along the top of the slope, introduced a public interface requiring careful management of safety, access, and communications during the works (Figure 3).

Overview of the Pollaphuca Erosion Remediation Works detailing Reservoir Erosion Works and associated Lessons Learned



Figure 3. Erosion at Burgage more and Avon Ri complex (Google maps, Google, 2026)



Figure 4. Revised Access Route (QS-000177-01-G451-004)

Early planning assessed alternative site access routes: (i) via private land north of the Avon Ri complex and (ii) via/through the Avon Ri complex. Minimising disruption initially favoured northern access. However, subsequent geotechnical assessment and constructability considerations informed a revised access strategy which meant that a route to the east and south was chosen for access (Figure 4).

INFORMATION SEEKING AND GROUND INVESTIGATIONS

At project initiation, available data comprised a prior borehole, bathymetric survey and drone survey. These datasets were reviewed and used to develop a 3D ground model (Civil 3D). However, it was determined that the existing borehole information was insufficient to define slope geometry changes and temporary works risks with adequate confidence.

A further ground investigation (GI) scope was developed and delivered during Winter 2021–2022 following a tender process. Works included boreholes at the crest to characterise materials to depth; dynamic cone penetrometer testing to assess the feasibility of the northern access; and dynamic probes/ window sampling along the shoreline and accessible slope locations.

GI execution required constrained shoreline working due to reservoir level fluctuations. Daily coordination with Pollaphuca Generation Station was required to align GI activities with planned reservoir levels. Shoreline works required restriction of reservoir levels to approximately 184.7 mOD for safe access. GI works were undertaken over a number of stints between January and July 2022.

The stratigraphy encountered was broadly consistent across the site and generally comprised topsoil, made ground overlying granular and cohesive deposits including laminated clays. Groundwater strikes were recorded where encountered, including standpipe recovery observations where feasible, and a full suite of geotechnical laboratory testing was completed to support subsequent analyses.

OPTIONEERING AND DESIGN DEVELOPMENT:

Following receipt of the GI factual report, assessment was carried out on (i) feasibility of the northern access route and (ii) remediation options for the unstable shoreline slopes. The superficial strata were predominantly relatively recent, unconsolidated glacio-lacustrine and possibly alluvial deposits, with laminated cohesive deposits encountered at greater depth interpreted as glacio-lacustrine in origin. These materials presented stability and constructability challenges for regrading, and design development incorporated a 20m land sterilisation zone from the crest back onto Avon Ri lands.

Overview of the Pollaphuca Erosion Remediation Works detailing Reservoir Erosion Works and associated Lessons Learned

The northern access route was judged unlikely to be feasible due to a combination of steep gradients, low shear strengths, laminated clays, and indications of potential existing failure mechanisms. A revised access route was therefore developed via the Avon grounds (and assessed by environmental and planning teams for consistency with the original Section 5 exempted development position). A preferred scheme was developed incorporating:

- A constructed access route enabling plant access from the Avon grounds to the shoreline, subsequently retained as a revised pedestrian access route (Figure 5).
- Shoreline protection via a rock-armour toe berm to mitigate ongoing erosion.
- A groundwater cut-off drainage feature intended to reduce internal erosion mechanisms contributing to instability and ground loss (later adapted due to ecological findings).
- Crest regrading to reduce destabilising forces, enable drainage installation, and generate fill for toe works, while retaining existing slope vegetation where practicable.
- Access control/ sterilisation of residual risk areas, supported by temporary design work controls during the works.

The design accounted for major constraints including limitations on extension into the reservoir (construction flooding/ temporary works risk), limitations on encroachment at the slope crest (amenity impacts), and incomplete verification of shoreline ground conditions due to access limitations. An observational approach (in line with Eurocode 7 principles) was adopted to manage residual uncertainty and enable adaptive decision-making during construction as conditions were revealed.

A Landscape Architect was appointed to design landscaping that restored visual amenity and provided appropriate planting in this recreational setting adjacent to the Avon Ri complex.

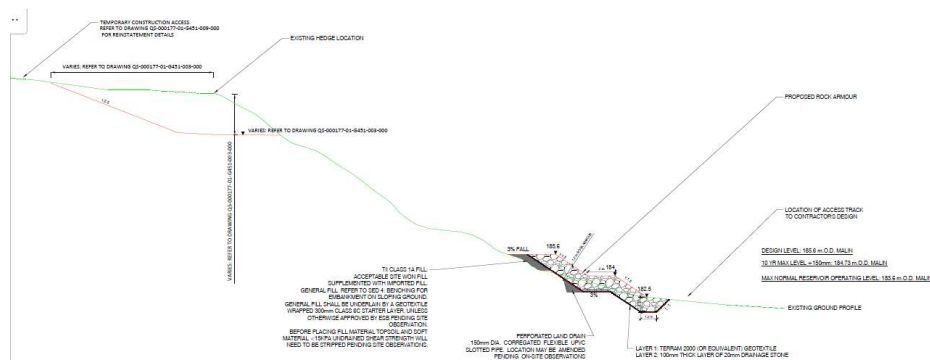


Figure 5. Cross Section of Works Proposed (QS-000177-01-G451-004)

PROCUREMENT AND TEMPORARY WORK RISK MANAGEMENT:

Tender documents were issued to framework contractors in May 2023, with tender return dates extended to facilitate site visits. A preferred tenderer was brought forward for post-tender engagement, however significant temporary works risk were highlighted associated with:

- Excavator and dump truck operations near the crest during bench excavation and drainage installation.
- Construction of a temporary crest-to-shore access route supporting heavy plant and rock delivery.
- Construction of a temporary shoreline access track to facilitate progressive placement of rock armour.

ESB and the preferred tenderer, William & Henry Alexander Ltd (WH Alexander), agreed that a temporary works design review should precede any construction contract award. ESB advised William & Henry Alexander Ltd on a suitable scope of works and a specialist geotechnical consultancy, Gavin Doherty Geo-Solutions (GDG) was appointed to complete the review and temporary works design.



Figure 6. View of slopes before works

Stage Excavation and Movement Monitoring

Temporary works stability analyses (Eurocode 7 DA1C2 and SLS checks) indicated the need for staged excavation and strict plant stand-off distances that reduced progressively as excavation advanced. The analysis considered 20 tonne plant, operating over a 7.5m wide works area, perpendicular from the crest of the slope. Physical demarcation was required to prevent plant encroachment into exclusion offsets.

Overview of the Pollaphuca Erosion Remediation Works detailing Reservoir Erosion Works and associated Lessons Learned

Survey monitoring stakes were specified in lines perpendicular to the crest (e.g., approximately 1m, 4m and 7m behind the crest) at defined chainage intervals, with survey frequency increased adjacent to active work zones (typically at the start and end of shifts). Trigger levels were defined using a simple “traffic light” approach; for example, works could proceed below a defined displacement threshold, while exceedance required immediate cessation, plant stand-down, and engineering review.

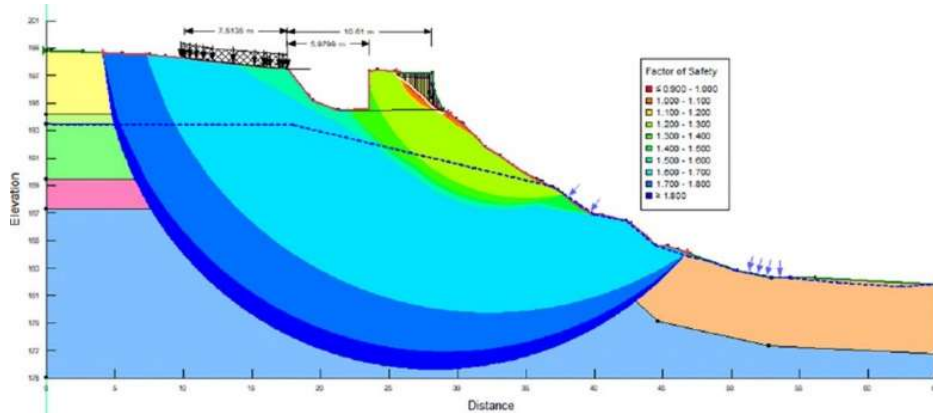


Figure 7. Phased approach to benching crest safely

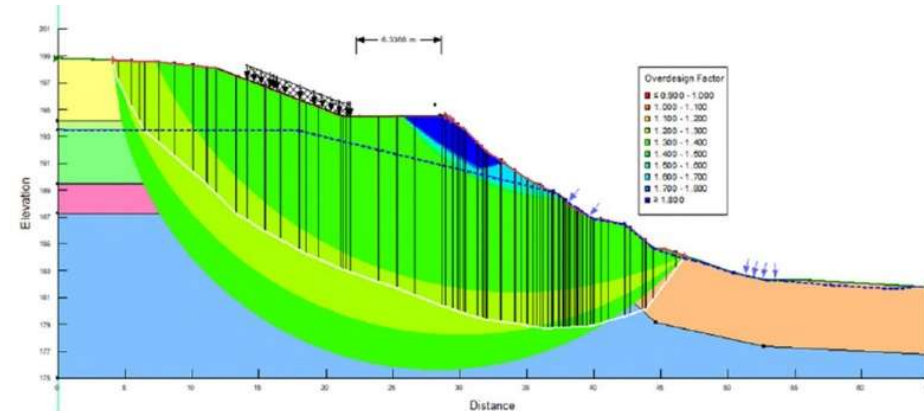


Figure 8. Phased approach to benching crest safely

Temporary Access Ramp and Shoreline Track

A temporary access ramp was required, with gradients steepening to approximately 18 % near the shoreline. The soils along the steeper portion of temporary access road were required to have a CBR of at least 1.5 %. The temporary works access road down the slope was recommended to comprise a 150 mm layer of 6F2 overlain by a 150 mm Unbounded Granular Mixture (UGM) A running layer with an intermediate geogrid. Continuous

monitoring of the downward access road was required to be undertaken to identify areas where topping up etc. would be required.

For shoreline access, the methodology integrated temporary access into the developing rock-armour footprint, enabling progressive extension of access as armour placement advanced. This approach improved constructability while controlling temporary stability risks. The temporary access was designed to be constructed using a starter layer of Class 6A material. Above the Class 6A starter layer, 400 mm of Class 6F2 engineered fill supporting a 150 mm UGM A capping layer with geosynthetic reinforcement of combigrid 40/40.

Following the temporary works design review by GDG, WH Alexander and ESB considered the approach recommended by GDG feasible. Following this review of temporary works, a review of the pricing of the contract was carried out by WH Alexander based on the changes considered as part of the temporary works approach. WH Alexanders were then appointed by ESB to carry out the construction works in September 2024.

CONSTRUCTION STAGE:

Construction commenced on site in November 2024. Early works included mobilisation and compound establishment; however, there was delay in securing access to lands used for compound and material storage. The Blessington Greenway was closed for a period up to the Christmas break to facilitate construction of the access track and associated works

Environmental constraint: identification of tufa springs

During site attendance by the project ecologist, seepages/ springs were identified and subsequently confirmed as tufa (petrifying) springs, described as lime-rich discharges capable of depositing calcium carbonate and forming porous tufa deposits. Tufa springs are designated as a priority habitat in Annex I of the European Union Habitats Directive in recognition of their vulnerability. A specialist was engaged to confirm habitat identification. Given the sensitivity of these features, a hydrogeological review assessed potential impacts of the proposed drainage and revised slope geometry, drawing on the borehole and groundwater dataset. The drainage design was modified, including raising a portion of the cut-off drain, to reduce risk of adverse effects.

Overview of the Pollaphuca Erosion Remediation Works detailing Reservoir Erosion Works and associated Lessons Learned



Figure 9. Tufa formation on reservoir shore



Figure 10. Original depth of filter drain which had to be subsequently raised to avoid tufa seepage zone

Programme adaptation: toe works executed ahead of crest regrading

Due to reservoir levels and the design review outcomes, programme sequencing changed. Rock armour was installed at the shoreline toe earlier than originally anticipated, with subsequent use of regraded crest material to fill and tie-in behind the armour. Some modifications to toe geometry were implemented during delivery to reflect constructed conditions.

Staged excavation at the crest proceeded with monitoring in accordance with the temporary works design. The temporary works designer undertook site visits during crest excavation and raised no concerns. It was considered that completing the toe berm early likely provided beneficial confinement and improved stability during subsequent crest works.

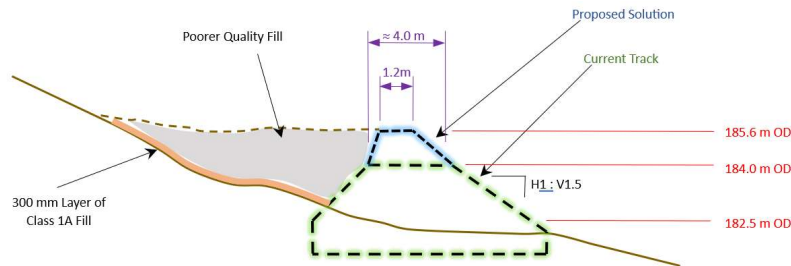


Figure 11. Geometry of rock armour toe incorporating shoreline access track (ESB,2025)

Permanent works and reinstatement

At crest level, permanent public interface works included a paved path, 1.5m high fencing and new hedge planting. Landscaping and planting were completed downslope of the fence line to reinstate and enhance visual amenity. No excess material disposal off-site was required during the works. The constructed access track was agreed to remain in place for future access requirements. The works were substantially completed by May 2025.

OUTCOMES, SUCCESSES AND LESSONS LEARNED

Delivery Outcomes

The final scheme provided shoreline protection and improved slope configuration at a high-sensitivity recreational location, while accommodating operational constraints associated with reservoir level fluctuations and difficult shoreline access. The methodology integrated temporary access and permanent shoreline protection in a manner that supported safe construction sequencing.

Success Factors

Key success factors included:

- Effective collaboration between ESB and the contractor to integrate shoreline access into the rock-armour solution.
- A structured temporary works approach that reduced geotechnical risk during excavation and access construction.
- Favourable reservoir levels during critical phases of shoreline works.

Overview of the Pollaphuca Erosion Remediation Works detailing Reservoir Erosion Works and associated Lessons Learned

Lessons Learned

The project highlighted several lessons applicable to similar reservoir perimeter remediation works:

- Earlier and more structured stakeholder engagement can reduce delays and improve access/ compound planning (particularly where third-party lands are required).
- Ecological constraints may be difficult to identify during early assessments; tufa springs were only identified due to local ecological knowledge during construction-stage site attendance. Future projects should consider targeted surveys during appropriate seasons and include contingency for habitat discoveries.
- Temporary works risk can be a determining factor in contractor appetite and tender responsiveness; early temporary works design development may improve market engagement and pricing certainty.
- Flexible sequencing (e.g., toe works ahead of crest regrading) can de-risk stability and maintain progress where reservoir levels or ecological findings require design changes.

CONCLUSIONS

Erosion and instability at Pollaphuca Reservoir continue to present technical and stakeholder challenges due to the interface between geotechnically complex shoreline deposits, fluctuating water levels, public amenities and environmentally designated features. At Burgage More, the adopted remediation approach, crest regrading combined with toe rock armour and adapted groundwater control measures, was delivered successfully using a staged and monitored temporary works methodology and an observational approach to manage uncertainty in shoreline conditions.

The project demonstrates the importance of integrating temporary works considerations into early design and procurement strategy, and the value of maintaining programme flexibility to respond to environmental constraints and water level conditions. The lessons identified are transferable to other reservoir shoreline erosion sites where constructability, public interface and habitat sensitivity are key drivers.



Figure 12. Aerial view of completed works



Figure 13. Aerial view of completed works

REFERENCES

National Parks and Wildlife Services, (2023) Guidelines for the Assessment of Annex I Priority Petrifying Springs. In *Ireland Irish Wildlife Manuals* 142

**Overview of the Pollaphuca Erosion Remediation Works detailing
Reservoir Erosion Works and associated Lessons Learned**

ESB EMP (2023) *QS-000177-01-R476-002-000 Pollaphuca Reservoir – Slope
Remediation at Burgage More - Design Report*

Gavin Doherty Geosolutions (2024) *24020-R-001-02 Slope Remediation
Temporary Works Design Report – Pollaphuca Reservoir*

ESB EMP (2023) *QS-000177-01-S165-002-000 Pollaphuca Reservoir
Remediation – Burgage More Scope of Works*

ESB EMP (2023) *Construction Drawings Package QS-000177-01-G451-002 to
018*

ESB International (2016) *QS-000177-01-R460-016 Section 5 Declaration
Application to Wicklow County Council*

ESB International (2016) *QS-000177-01-D460-013 Site Location Map
Burgage More*

<https://esb.ie/what-we-do/generation-and-trading/poulaphuca---hydro>
(April 2026)



The British Dam Society

The factor of safety in finite element analysis of embankment dams

N KOVACEVIC, Geotechnical Consulting Group
D M POTTS, Imperial College London

SYNOPSIS

New reservoirs in the UK will be retained by earth embankment dams constructed on stiff plastic, over-consolidated and low permeability clay formations with relatively low shear strength, which may further reduce due to post-peak strain-softening leading to the mechanism of progressive failure. In practical stability analysis of these dams the Factor of Safety (FoS) is usually determined by means of simple Limit Equilibrium (LE) analysis which cannot account for this mechanism. Advanced Finite Element (FE) analysis which are nowadays routinely used for assessing displacements and stresses in embankment dams can. However, the determination of a FoS within a FE analysis is a more complex issue. It is usually carried out by means of the so-called strength reduction method, but different software programs account for this in different ways, and mostly only for simple constitutive models.

This paper briefly outlines a consistent methodology for accounting for the FoS in FE analysis, which can be applied to any constitutive model, even those accounting for strain-softening and progressive failure. It then describes a series of FE analyses undertaken to determine the operational FoS during the various life stages of the main embankment dam at Abberton Reservoir which was raised from 2009 to 2013. The original slip in the upstream slope of the dam in 1937 through the mechanism of progressive failure provided reliable data to determine the strength of the various materials involved.

INTRODUCTION

The design of embankments has presented stability problems in the past, particularly when high embankments (i.e. dams) on undrained (clay) foundations were attempted (Briddle et al, 1985). The application of soil mechanics principles led to design in terms of pore pressure and effective stress, with stability analysis by LE methods. However, owing to the inherent assumptions in LE analysis this method does not always provide a unique FoS. Also, stiff over-consolidated and plastic clay foundations (like many beneath newly proposed reservoirs in the UK) may involve a reduction in shear strength due to post-peak strain-softening leading to the mechanism

The factor of safety in finite element analysis of embankment dams

of progressive failure. LE analysis cannot account for this mechanism (Vaughan et al, 2004).

Design of these dams has changed with the development of advanced numerical methods, such as the FE method of analysis (Kovacevic et al, 2008). Apart from assessing displacement and stresses under working load conditions, this technique is now increasingly being used to calculate the FoS, and results are reported to be comparable to those obtained with LE methods (e.g. Brinkgreve & Bakker, 1991; Griffiths & Lane, 1999).

However, the inclusion of a FoS within an advanced FE analysis is a complex issue. There is no guidance on the appropriate numerical procedure that should be implemented, and different software programs account for this in different ways, and mostly only for simple constitutive models of the Mohr-Coulomb and Tresca type. Such procedures can be problematic, particularly for more complex constitutive models, however these models are often required to predict the realistic behaviour of embankment dams during both construction and subsequent operation (Kovacevic et al, 2011).

Potts & Zdravkovic (2012) illustrated some of the difficulties that may arise when attempting to calculate a FoS from FE analyses. For more complex boundary value problems, such as embankment dams on undrained foundations subject to progressive failure, and when using advanced constitutive models, these difficulties may increase. They developed a consistent methodology to account for the FoS in FE analyses which can be applied to any constitutive model, even those accounting for strain-softening and progressive failure. In this paper this methodology is used to estimate the stability and operational FoS during different stages in the life of the main embankment dam at Abberton.

FACTOR OF SAFETY BY STRENGTH REDUCTION METHOD IN FE ANALYSIS

In FoS FE analyses the 'material' strength used is based on the 'design' strength but reduced by a FoS. For analyses based on undrained strength, S_u , the strength used in an analysis, $S_u^{analysis}$, is estimated as:

$$S_u^{analysis} = S_u^{design} / \text{FoS}$$

whereas for analyses based on drained strength, ϕ' , the strength used in an analysis, $\phi'^{analysis}$, is estimated as:

$$\phi'^{analysis} = \tan^{-1}(\tan \phi'^{design} / \text{FoS})$$

where S_u^{design} or ϕ'^{design} are the characteristic values of soil strength best estimated from the available site investigation data.

The above strength reduction can be achieved in two ways in a FE analysis. The first approach is to use the designed strength (S_u^{design} and ϕ'^{design}) directly without modification, and then at relevant stages of the analysis to gradually increase the FoS (i.e. to reduce the design strength) until failure occurs. The second approach is to start the analysis with the factored strength (S_u^{analysis} or ϕ'^{analysis}) and continue until failure occurs.

The first approach has the advantage that a single FE analysis could be used to investigate both serviceability and ultimate limit states. This is especially true for the embankment dams which go through many different life stages. Given that now there is a unique way of how the strength reduction is numerically implemented irrespective of the constitutive models used in the FE analyses (Potts & Zdravkovic, 2012) this approach appears to be particularly attractive in the calculation of the FoS.

The advantage of the second approach is that no modification to the analysis software is needed, which makes it an easier option and can be used with most constitutive models. However, the disadvantage is that such a reduced design strength may require initial stresses that are not consistent with those in-situ. In addition, the various stages of the analysis may be completed without reaching failure, which ensures the stability of the problem, but does not produce information on the magnitude of the FoS.

The proposed unique procedure behind the first strength reduction approach irrespective of the constitutive model used in the FE analyses is presented in Potts & Zdravkovic (2012). They introduced a FoS as an additional state parameter in the yield function of the constitutive model, and then the necessary modifications to the FE formulation in terms of governing equations and the relationship between changes in stresses and strains appear to be relatively straightforward.

This novel approach in calculating the FoS has been used in all FE analyses presented in this paper. These analyses have been performed with the FE software ICFEP (Potts & Zdravkovic, 1999) in two dimensions using a plane strain approach and eight-noded FE elements with reduced 2x2 integration order. A modified Newton-Raphson non-linear solver with an error-controlled sub-stepping stress-point algorithm was used.

MAIN EMBANKMENT AT ABBERTON RESERVOIR

The 'original' embankment at Abberton has a narrow central vertical core of puddle clay which extends below ground level to form a cut-off trench through the underlying Alluvium and into the in-situ London Clay (see the 'original' dam geometry in Figure 1). Transition zones of selected clay fill

The factor of safety in finite element analysis of embankment dams

were placed between the central clay core and outer portions of the upstream and downstream embankment shoulders.

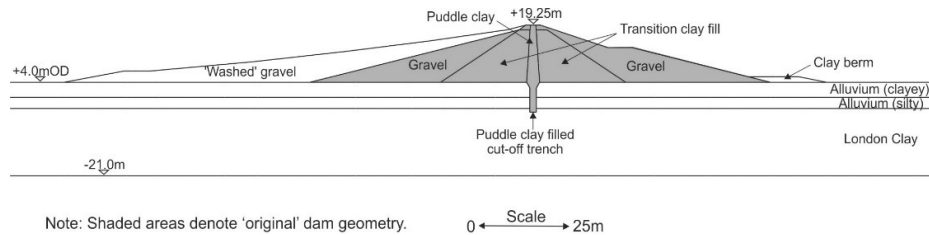


Figure 1. Zoning of 'original' and 're-constructed' dam

The FE analyses modelled the failure during 'initial' dam construction in 1937 (Figure 2) and its subsequent successful completion and operation (Kovacevic et al, 2013), before it was raised from 2009 to 2013 (Figure 3). The best estimate of the dam history was modelled in the 'coupled' FE analyses. Table 1 shows the increment numbers corresponding to the various stages of the analysis and appropriate time steps. The model parameters were derived on the basis of the available site investigations and laboratory testing (Table 2).



Figure 2. General view of 'original' dam failure

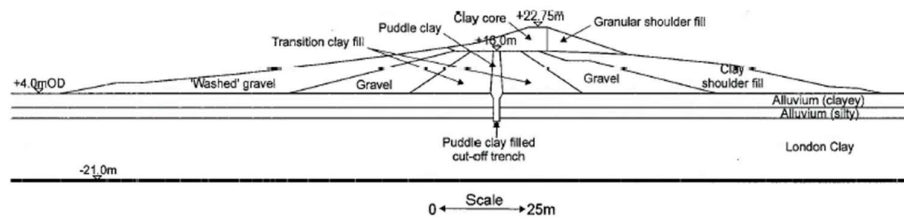


Figure 3. Zoning of raised dam

Table 1. Construction sequence modelled by the FE analysis

Inc.No.	Description of event	Time
0	Set-up initial stresses	-
1...5	Cut-off trench excavation	4.5mths
6...10	Cut-off trench filling	4.5mths
11...23	'Original' embankment filling to El.+18.0mOD	7.2mths
24...30	Removal of 'failed' material to El.+9.0mOD	2.52mths
31...41	Filling for 're-constructed' embankment to El.+19.25mOD	6.6mths
42...50	First reservoir impounding to TWL (El.+18.0mOD)	33mths
51	Reservoir drawdown to El.+16.25mOD	6mths
52, 53	Dissipation of p.w.p. with reservoir level at El.+16.25mOD	5.75yrs
54	Reservoir re-impounding to TWL (El.+18.0mOD)	6mths
55...60	Dissipation of excess p.w.p. at TWL (El.+18.0mOD)	40yrs
61...64	Reservoir drawdown to El.+15.0mOD and re-impounding	1yr
65, 66	Reservoir drawdown to El.+16.25mOD and re-impounding	1yr
67...70	Dissipation of excess p.w.p. at TWL (El.+18.0mOD)	20yrs
71...79	Filling for 'raised' embankment to El.+16.0mOD	10.8mths
80, 81	Reservoir drawdown to El.+15.0mOD	0.25mth
82...84	Removal of 'existing' dam crest to El.+16.0mOD	1mth
85...87	Filling for 'raised' embankment to El.+19.25mOD	3.6mths
88, 89	Reservoir impounding to 'old' TWL at El.+18.0mOD	0.25mths
90, 91	Filling for 'raised' embankment to El.+22.75mOD	2.4mths
92...94	Reservoir impounding to 'new' TWL at El.+21.0mOD	6mths
95...100	Dissipation of p.w.p. to establish long-term conditions	100yrs

'Initial' dam construction began in March 1936, and the upstream embankment shoulder of the 'dam suffered a deep-seated failure through its foundation in July 1937 when construction of the embankment dam was still some 2m short of its full height (Figure 4).

The FE analysis predicted the failure of the upstream shoulder during the placement of the layer of fill between +17.25 and +18.0mOD, i.e. approximately 2m from the intended crest level at +19.5mOD. Results are presented for the last three stages of the analysis (just prior to and during failure) in terms of the incremental movement vectors and accumulated contours of plastic shear strain (Figure 5). The clear progression towards failure can be seen.

The factor of safety in finite element analysis of embankment dams

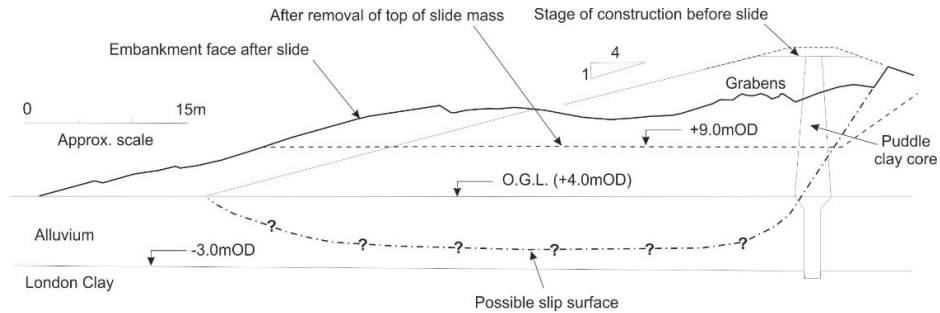


Figure 4. Failed section of the 'original' dam (after Watson Hawksley, 1990)

Table 2. Material properties assumed in the analysis

Material property	Puddle Clay*	Transit-ion Fill*	General Gravel	Wash-ed Gravel	Clay All-uvium	Silty All-uvium	In-situ London Clay
Unit weight γ (kN/m ³)	19	18	20	20	18	18	19.5
Strength	$\phi'_{cr}=21^\circ$	$\phi'_{cr}=23^\circ$	$c'=1\text{kPa}$ $\phi'=35^\circ$ $\psi=10^\circ$	$c'=1\text{kPa}$ $\phi'=35^\circ$ $\psi=10^\circ$	$c'=5\text{kPa}$ $\phi'=20^\circ$ $\psi=0^\circ$	$c'=3\text{kPa}$ $\phi'=30^\circ$ $\phi=0^\circ$	$c'_p=8\text{kPa}$ $\phi'_p=24^\circ$ $c'_r=2\text{kPa}$ $\phi'_r=13^\circ$ $\psi=0^\circ$
Stiffness	$\lambda=0.113$ $\kappa=0.030$ $\mu=0.30$	$\lambda=0.113$ $\kappa=0.030$ $\mu=0.30$	$E=200.$ $p'(\text{kPa})$ $E_{min}=5$ MPa $\mu=0.20$	$E=200.$ $p'(\text{kPa})$ $E_{min}=5$ MPa $\mu=0.20$	$E=65.p'$ $+5200$ (kPa) $\mu=0.30$	$E=65.p'$ $+5200$ (kPa) $\mu=0.30$	$E=5000.(p'$ $+100)/100$ (kPa) $\mu=0.20$
Permeability k (m/s)	3×10^{-11}	1×10^{-9}	-	-	$1 \times 10^{-8}.$ $e^{-0.002.p'}$	1×10^{-7}	$5 \times 10^{-10}.$ $e^{-0.007.p'}$

(*) – Specific volume at $p' = 1\text{kPa}$ is $v_1 = 1 + e_1 = 2.5$ in the Modified Cam Clay model which was used for the characterisation of the puddle clay and transition fill. Note that the Mohr-Coulomb model is used for all other materials. For the in-situ London Clay shear strength parameters are reducing from peak values (c'_p and ϕ'_p) at $\epsilon^p_{Dp}=5\%$ to residual values (c'_r and ϕ'_r) at $\epsilon^p_{Dr}=20\%$, where ϵ^p_D is the plastic shear strain invariant defined as $(\epsilon^p_D)^2 = (2/3) \cdot [(\epsilon^p_1 - \epsilon^p_2)^2 + (\epsilon^p_2 - \epsilon^p_3)^2 + (\epsilon^p_3 - \epsilon^p_1)^2]$.

The FE analysis of the 'initial' failure confirmed the deep-seated progressive failure mechanism postulated by French et al (2000), which involved shearing through the top of the London Clay foundation rather than a non-circular failure mechanism through the alluvium, as advocated by Watson Hawksley (1990) and indicated in Figure 4. However, the predicted rupture surface appeared to be somewhat deeper (3m below the top of London Clay) than expected, because the top 2m of the London Clay gained strength

owing to its consolidation into the overlying more permeable alluvium (Figure 6a). By definition, for this stage of the analysis the $FoS=1.0$.

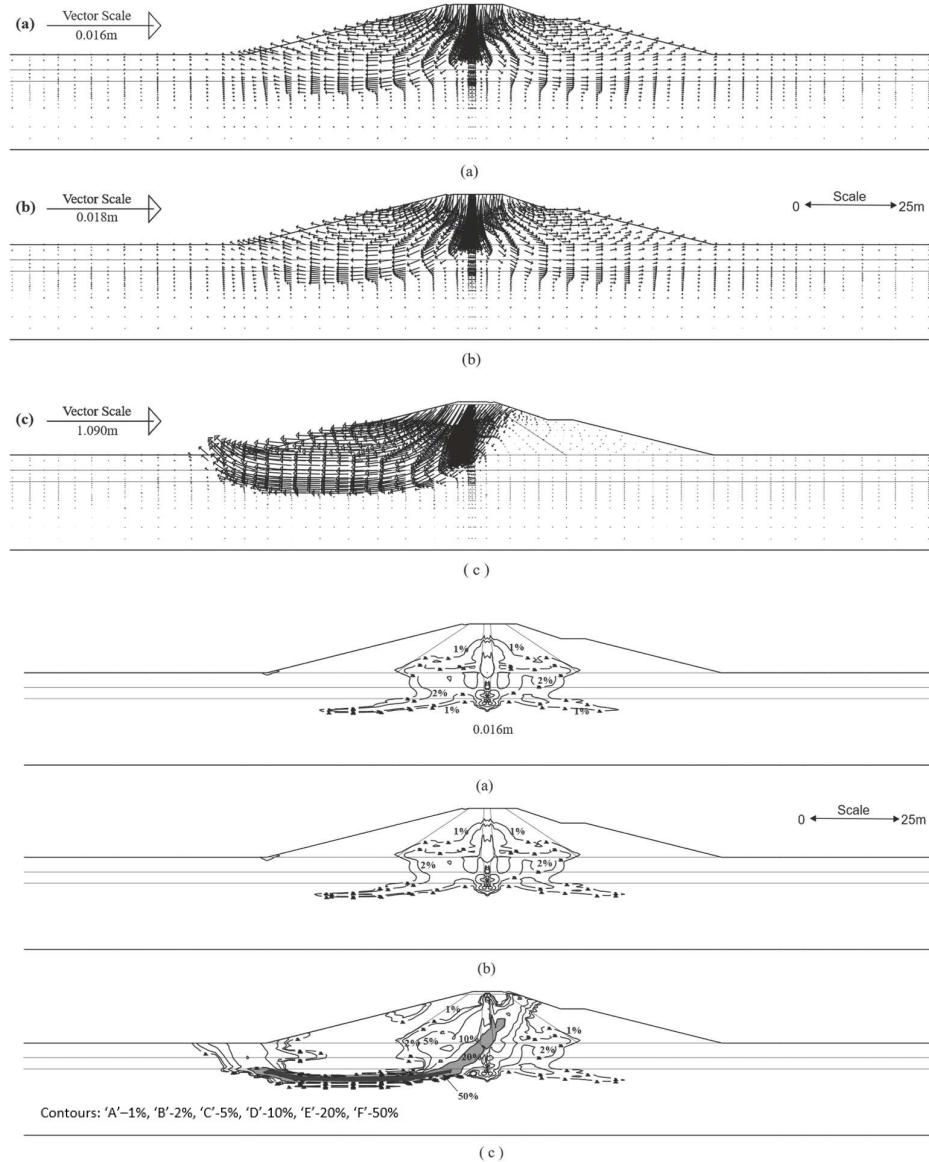


Figure 5. Predicted incremental movement vectors and contours of plastic shear strain towards the end of construction of the 'original' dam

RESULTS OF FACTOR OF SAFETY ANALYSES

In the FE analyses, the FoS was incrementally increased from an initial value of 1.0 until the factored strength resulted in an unstable slope. Typically, the increase was in 10% (i.e. 0.10) increments, and when failure was approached

The factor of safety in finite element analysis of embankment dams

the increments reduced in size to just 1% (i.e. 0.01). It should be noted that the time and associated pore pressure changes were stopped after the increment prior to the first FoS increment and the remaining FoS analysis was drained. In each increment of the FoS analysis, it is the current strength that was factored meaning that the strain-softening behaviour was captured by the analyses.

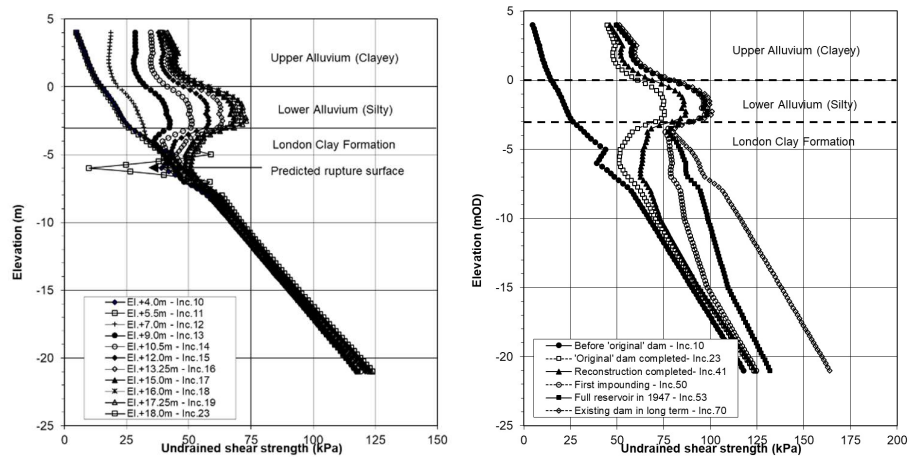


Figure 6: Predicted undrained shear strength profiles in the dam foundation at (a) upstream and (b) downstream 32.5m from the dam centreline

The results are presented in terms of:

- the contours of plastic shear strain invariant, ϵ^p_D (see definition in Table 2) just prior to the predicted failure,
- the velocity characteristics just prior to the predicted failure, which indicate the zones in which plastic yield is currently occurring (note that in the case of the Mohr-Coulomb model, which was used to characterise the behaviour of the various foundation materials and gravel fills, they indicate the areas where the shear strength is fully mobilised), and
- the incremental movement vectors during the failure; as such, they indicate the position of the predicted failure surface, and their magnitude is not of importance.

The proposed strength reduction method for determination of the FoS can be applied for any stage (increment) of the FE analysis as outlined in Table 1. The following four stages in the life of the Abberton dam after its 'original' failure are considered herewith:

- the end of construction of the re-constructed dam (Inc.41 in Table 1),
- after dissipation of the excess pore pressures after dam re-construction in the long-term, before the dam raising (Inc.70),
- after full reservoir impounding after the dam raising (Inc.94), and

- after dissipation of the excess pore pressures after dam raising in the long-term (Inc.100).

The re-constructed dam, after the 'original' dam failure, had a flatter upstream slope (Figure 1) and the FE analysis predicted that the steeper downstream slope is now more critical (Figure 7). The time needed for re-construction contributed to a strength increase in the London Clay, resulting in a deeper failure surface being predicted beneath the downstream slope (Figure 7) than beneath the 'original' upstream slope (Figure 5). The predicted FoS for the re-constructed dam is 1.30. There is no indication that strain-softening (note that there is no 'C' contour beneath the downstream slope, which indicates plastic shear strains in excess of 5% and strain-softening, see Table 2) prior to failure is involved in the London Clay foundation (Figure 7a). Furthermore, it appears that the shear strength is not yet mobilised beneath the downstream toe just prior to failure (Figure 7b). It is worth mentioning that LE stability analyses cannot account for this latter mechanism of behaviour.

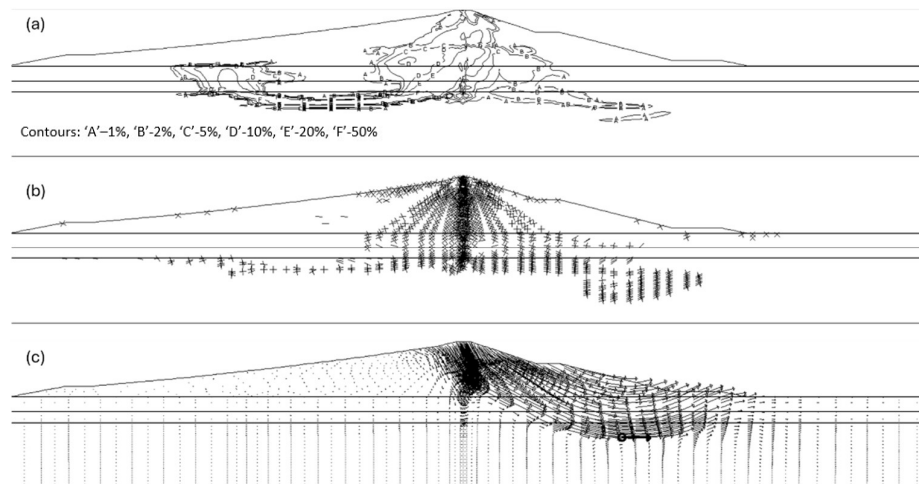


Figure 7. The end of construction of the 're-constructed' dam (FoS=1.30)

Dissipation of the excess pore pressures in the foundation of the re-constructed dam, to just before dam raising, caused a further increase in the foundation strength (Inc.70 in Figure 6b). The FE analysis predicted a 'shallower' critical failure surface beneath the downstream slope of the re-constructed through the 'weaker' clay alluvium (see Table 2). The predicted FoS in the long-term is increased to FoS=1.57 (Figure 8).

The factor of safety in finite element analysis of embankment dams

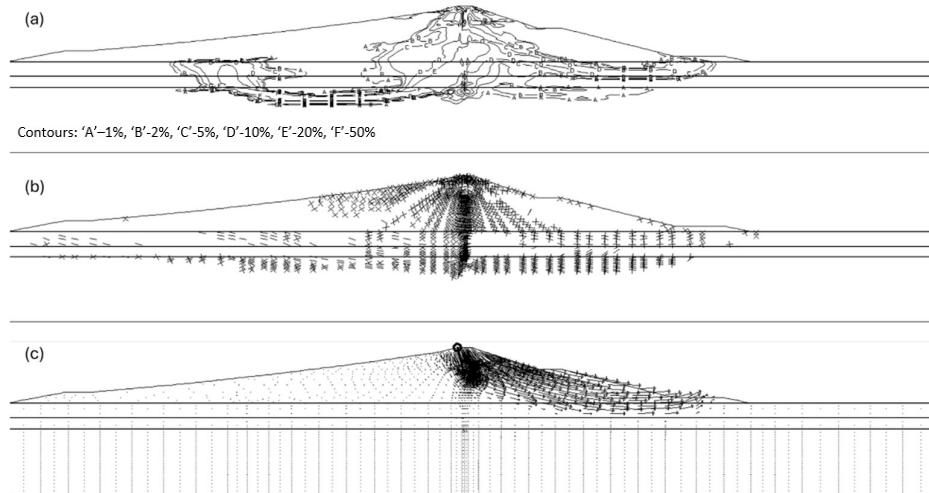


Figure 8. Re-constructed dam in the long-term (FoS=1.57)

The increased foundation strength in the long-term was sufficient to allow safe construction during the subsequent dam (and reservoir) raising. The dam is raised downstream (Incs 71 to 94, see Table 1) and has a flatter downstream slope than the re-constructed dam. The predicted failure surface at the end of raising is deep-seated through the London Clay foundation (Figure 9c), without any significant sign of progressive failure prior to collapse (Figure 9a), and the predicted short-term FoS is 1.70.

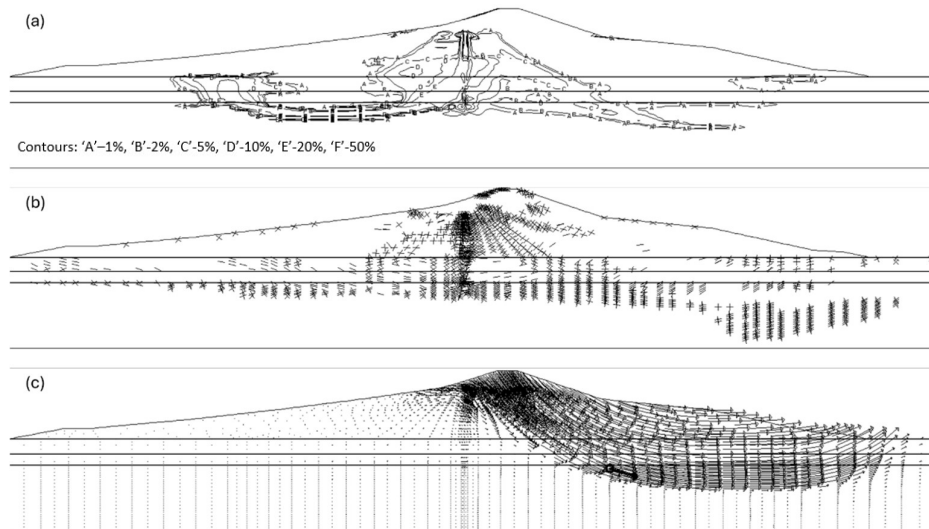


Figure 9. Full reservoir impounding after the dam raising (FoS=1.70)

Finally, the FE analysis modelled dissipation of the excess pore pressures after the dam raising to the long-term (Incs 95 to 100, Table 1), the London Clay gained further strength, and the failure surface is again predicted to

pass through the 'weak' clay alluvium. The predicted FoS in the long-term is increased to 2.02.

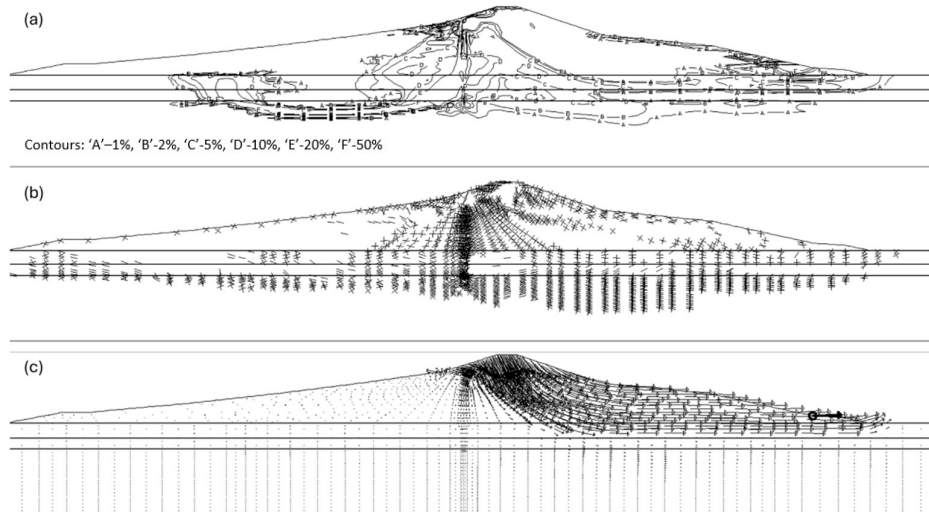


Figure 10. Raised dam in the long-term (FoS=2.02)

CONCLUSIONS

Nowadays, FE analyses are not only used to assess displacements and stresses in embankment dams, but they can also provide reliable values of FoS using a consistent methodology. This is irrespective of the constitutive model used to characterise the various materials involved and also accounting for possible strain-softening and the mechanism of progressive failure. Furthermore, the FE analyses can provide a FoS for any stage during the life of the dam showing realistic mechanisms of behaviour. This has been demonstrated on the example of the main embankment dam at Abberton Reservoir which was recently successfully raised. The predicted FoS were satisfactory for the four chosen stages of the analysis, both in the short and long-term.

REFERENCES

- Brinkgreve R B J and Bakker H L (1991). Non-linear finite element analysis of safety factors. In *Proc. 7th Int. Conf. Computer Methods and Advances in Geomechanics*, Cairns, 1117-1122.
- Bridle R C, Vaughan P R and Jones H N (1985). Empingham Dam - Design, construction and performance. *Proc. Inst. Civ. Engrs*, London, Part 1, Vol.78, 247-289.
- French D J, Woolgar M J and Saynor P (2000). Geotechnical investigations at Abberton Dam, Essex. In *Dams 2000. Proceedings of the 11th British Dam Society Conference*, Thomas Telford, London, 345-359.

The factor of safety in finite element analysis of embankment dams

- Griffiths D V and Lane P A (1999). Slope stability analysis by finite elements *Geotechnique*, **49(3)**, 387-403.
- Kovacevic N, Potts D M and Vaughan P R (2008). Recent advances in the numerical modelling of embankment dams. In *Ensuring Reservoir Safety into the Future. Proceedings of the 15th Conference of the British Dam Society*, Thomas Telford, London, UK, 164-176.
- Kovacevic N and Potts D M (2011). Use of advanced constitutive models in numerical analysis of embankment dams. *Proc. 2nd Int. Symp. Computational Geomechanics (ComGeo 2011)*, Cavtat, Croatia.
- Kovacevic N, Hight D W, Potts D M and Carter I C (2013). Finite element analysis of the failure and reconstruction of the main dam embankment at Abberton Reservoir, Essex, UK. *Geotechnique*, **63(9)**, 753-767.
- Potts D M and Zdravkovic L (1999). *Finite element analysis in geotechnical engineering: Theory*. Thomas Telford, London.
- Potts D M and Zdravkovic L (2012). Accounting for partial material factors in numerical analysis. *Geotechnique* **62(12)**, 1053-1065.
- Tschuchnigg F, Schweigwer H F, Sloan S W, Lyamin A V and Raissakis I (2015). Comparison of finite-element limit analysis and strength reduction techniques. *Geotechnique* **65(4)**, 249-257.
- Vaughan P R, Kovacevic N and Potts D M (2004). Then and now: some comments on the design and analysis of slopes and embankments. *Proc. Skempton Conference*, Thomas Telford, Vol.1, 241-290.
- Watson Hawksley (1990). *Abberton Dam: embankment stability and settlement. Review Report*.



The British Dam Society

Some considerations in assessing hydraulic uplift risk for large reservoirs on overconsolidated clays

N A MAPPERLEY, Mott MacDonald

M R J MULHOLLAND, Mott MacDonald

T BLOWER, *formerly* Mott MacDonald (retired)

SYNOPSIS

The UK currently has a number of major reservoirs in planning, design development or construction, all located on stiff clay strata of various ages. Where such strata are underlain by regional aquifers with artesian groundwater pressures, a risk of groundwater uplift may exist for the case where the borrow pit has been excavated but not yet filled.

Such is the case at the site of Anglian Water's proposed Lincolnshire Reservoir. Therefore, as part of the early design development, studies have been undertaken to assess the uplift risk for the base of this reservoir. The paper describes the geological and hydrogeological background of the site, the methodology applied in assessing hydraulic uplift risk, and presents and discusses the key assessment results.

INTRODUCTION

The drive to construct new reservoirs to improve the UK's water resilience is gathering pace. The sites for new reservoirs will be, or have been, selected based on a range of criteria including geographical, hydrological, social, environmental, cost and other grounds, and the resulting selected sites may have significant ground risks. Given the lack of reservoir building in the UK in recent decades, and scale of strategic reservoirs being planned, some of these challenges may be new or at least of a scale unprecedented to today's generation of reservoir engineers. For sites with artesian groundwater, groundwater uplift of the base of the reservoir may be one such case in point. Such occurrences are not merely theoretical: Moore and Longworth (1978) provide an example where base failure of a large excavation in Oxford Clay did indeed occur.

As part of the early design development for Anglian Water's two flagship reservoirs in the east of England, Fens Reservoir and Lincolnshire Reservoir, studies have been undertaken to assess the uplift risk for the base of the reservoirs. The construction of these reservoirs will involve the excavation of substantial volumes of clay bedrock to considerable depths. Both sites overlie aquifers with artesian groundwater levels of the order of 10 m above

Some considerations in assessing hydraulic uplift risk for large reservoirs on overconsolidated clays

ground level. Thus, whilst the natural landscape may be stable, the new excavated ground configuration is at greater risk, especially when not balanced by the weight of water in the reservoir.

The paper focuses on an early assessment undertaken for Lincolnshire Reservoir in support of Environmental Impact Assessment and Development Consent Order documentation.

BACKGROUND

Brief descriptions of the geology and hydrogeology of the Lincolnshire Reservoir site are provided below.

Geological Setting

The bedrock stratigraphical sequence underlying the site is presented in Figure 1. The geology generally comprises varying thicknesses of Glacial Till over bedrock comprising the Oxford Clay Formation (OXC). The OXC is underlain by: Kellaways Formation (Kellaways Sand (KLS) and Kellaways Clay (KLC)), Cornbrash Formation (CB), Blisworth Clay Formation (BWC), Blisworth Limestone Formation (BWL), Rutland Formation (RLD), and Lower Lincolnshire Limestone (LL). A West to East cross section of the stratigraphy across the centre of the site is provided in Figure 2.

Hydrogeological Setting

The hydraulic uplift force acts at the top of a confined aquifer (base of the confining layer). There are several aquifer and aquitard units underlying the Oxford Clay at the Lincolnshire Reservoir site as shown in Figure 1. The main ones of interest are the Kellaways Sand member, given this aquifer directly underlies the Oxford Clay and holds water in artesian or sub-artesian pressure, and the Lincolnshire Limestone, a Principal Aquifer used by Anglian Water for public water supply, which holds water in artesian pressure and is the aquifer that feeds the pressure of the whole system, being the aquifer with the highest piezometric level and greatest transmissivity in the system.

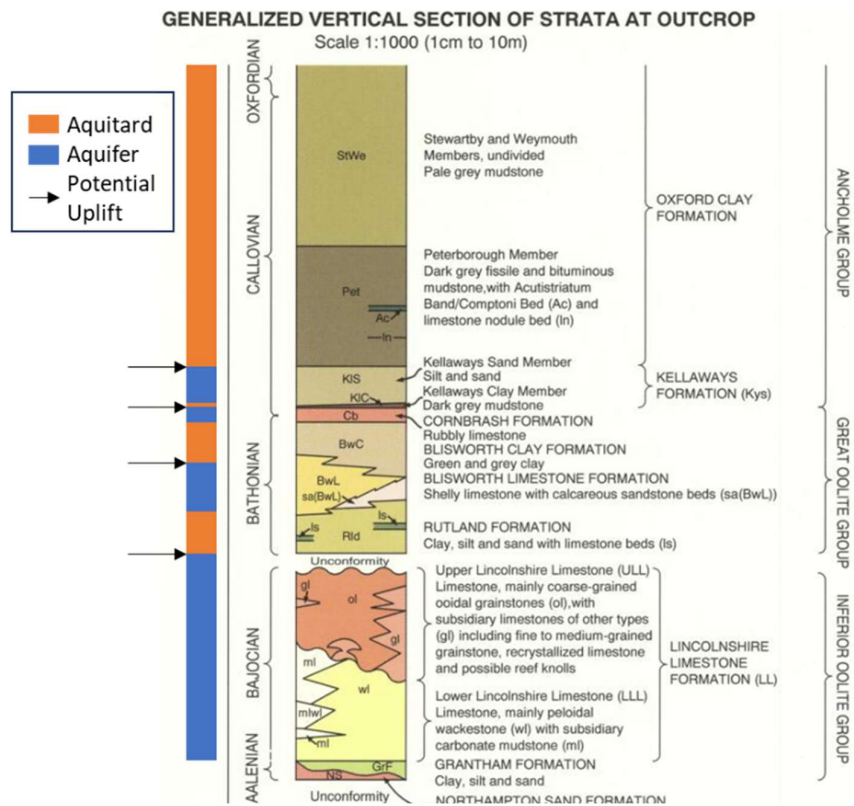


Figure 1. BGS bedrock section showing aquifer units and levels of potential hydraulic uplift. Reproduced with the permission of the British Geological Survey © UKRI [2026]. All Rights Reserved.

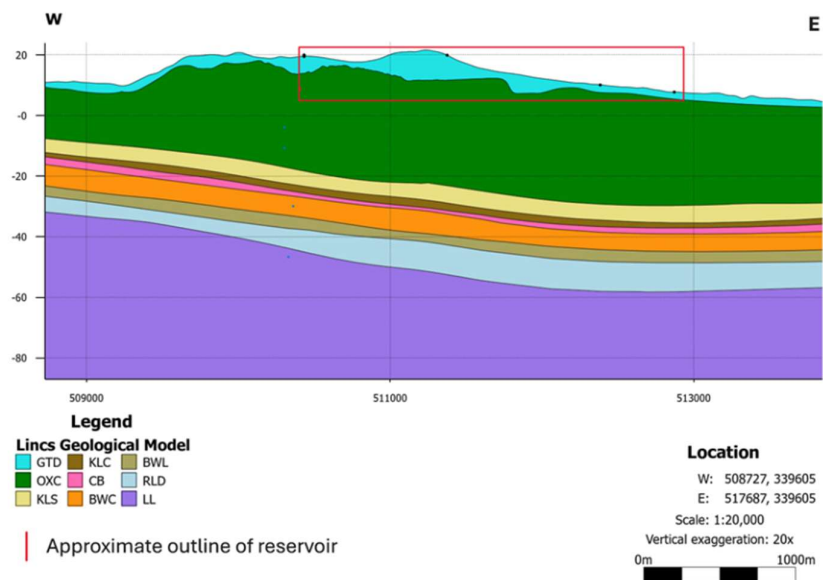


Figure 2. West to East stratigraphy with approximate reservoir location marked.

Some considerations in assessing hydraulic uplift risk for large reservoirs on overconsolidated clays

METHODOLOGY

Principles

Hydraulic uplift can occur when the upwards force of groundwater pressure at the base of impermeable strata is greater than the weight of overlying material. For an initial assessment, this force balance was considered by calculating a global Factor of Safety (FoS), where $FoS < 1$ indicates hydraulic uplift is expected to occur. The FoS was calculated using equation (1).

No other forces, such as the side friction of the uplifting mass, are considered since the size of the excavation means that shear strength resistance at the side of the lifting block is negligible compared to the uplift and weight forces.

$$FoS = \frac{\text{Weight of overlying soil and water}}{\text{Force from groundwater pressure head}} \quad (1)$$

$$FoS = \frac{(C-D)*\gamma_{soil} + (B-C)*\gamma_{water}}{(A-D)*\gamma_{water}} \quad (2)$$

Where γ_{soil} = soil bulk unit weight; γ_{water} = water bulk unit weight; and levels A, B, C, and D (mAOD) as indicated in Figure 3.

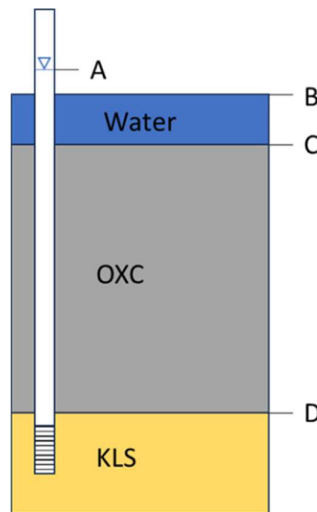


Figure 3. Schematic representation of elevations for determining the FoS against uplift.

Key Questions

In developing the assessment methodology, a number of key questions came to light that required resolution.

- Which horizon is the critical one for hydraulic uplift?
- What groundwater level should be taken for each design scenario?
- Is the risk of hydraulic uplift constant across the reservoir site?

Sources of groundwater data

Several sources of information were used to determine a representative 'piezometric level' or total head (level A in Figure 3) in the strata below the site. These include:

- (A) Environment Agency historic groundwater level data
 - Included 22 boreholes of relevance with 21 monitoring groundwater pressure in the LL and 1 installed in the KLS. Monitoring data typically covered from the 1970s to 2023. A maximum piezometric level of 21.5mAOD was recorded.
- (B) Lincolnshire Groundwater Numerical Model
 - A hydrogeological numerical model of the aquifers in the Lincolnshire area. A worst case scenario run of this model, where groundwater abstraction is ceased, was used to predict the groundwater total head. This predicted a maximum piezometric level in the LL within the reservoir site of 23.6mAOD.
- (C) Site specific groundwater monitoring data
 - Installations for the Lincolnshire Reservoir project. Groundwater level data measured in several vibrating wire piezometers (VWPs), covers a range of different stratigraphic units, including LL, BWL, CB, KWC, KWS, and OXC. Monitoring data was available between September 2023 and April 2025. The highest recorded piezometric level was 23.9 mAOD in the LL stratum.

Selection of a representative 'piezometric level' becomes complex where measured and modelled pore pressures show significant spatial and temporal variability. Eurocode 7 requires for ultimate limit states "design values shall represent the most unfavourable values that could occur during the design lifetime of the structure"; however, in practice, how should this be defined where groundwater heads fluctuate, exhibit localised extremes, and differ between observations and model outputs, particularly when considering the complex interaction between reservoir operating levels, construction programme and groundwater pressures? Then climate change also needs to be considered. For pragmatic reasons, therefore, a range of values have been applied to different scenarios for this initial assessment.

Standards of Assessment

There are several standards which could be applied to provide a suitable target FoS for uplift cases. Whilst Eurocodes do have a methodology, Eurocode 7 Part 1 states that for structures of the highest consequence class, *"additional or amended provisions to those given in EN 1997 (all parts) can be needed"*. Hence there can be uncertainty in which to apply.

Some considerations in assessing hydraulic uplift risk for large reservoirs on overconsolidated clays

USBR Design Standard 13 (Chapter 8) provides a target recommended global FoS for uplift of 2.0 for new dams and 1.5 for existing dams. As such, a FoS of 2.0 is required for most cases. However, a FoS of 1.5 may be considered more appropriate under extreme or accidental cases, or where there is strong rationale for adopting a lower FoS. Additionally, the USBR guidance is generally considered as a check for uplift risk at the toe of an embankment dam downstream slope. It is not considered directly applicable for a global uplift criterion for an excavation base.

The criteria used in the Eurocodes may possibly be more applicable to the type of uplift mechanism considered in this case. The mechanism for hydraulic failure due to uplift is set out in Section 10 of BS EN 1997-1:2004+A1:2013. The FoS against hydraulic failure due to uplift is calculated in the same way as described above in 'Principles'. However, as the Eurocode system applies a partial factors approach, an equivalent target global FoS has been calculated for comparison with the USBR standards.

The hydraulic heave limit state set out in BS EN 1997-1:2004+A1:2013 is applied in situations where differential water pressures cause the inflow of water. This limit state seems less directly applicable to the situation but was included for comparison. In summary, the various comparator target factors of safety are as follows:

- USBR (global FoS):
 - Recommended global FoS against uplift for new dams: 2.0
- Eurocode 7 (equivalent from partial FoS)
 - Uplift Limit State (UPL): 1.30
 - Hydraulic Heave limit state (HYD): 1.53

Scenarios for consideration

During and shortly after construction, the reservoir will be empty of water (level B in Figure 3 will be equal to level C). Construction will likely take place over a 3 to 4-year period. When the reservoir is in operation, the lowest realistic reservoir level will be during a 1 in 500-year drought. In this case, there will be 5 million m³ of water remaining in the reservoir, which will correspond to a water depth of 2.7m based on the Interim Design. This will be an 'unusual' event and will last, presumably, a matter of months.

The base of the reservoir (Level C in Figure 3) is dependent on several factors. On a simplistic basis though, the deeper the base of the reservoir the greater the achievable storage capacity which is a benefit to the Client. At interim design stage a level of 5.0mAOD was taken. A summary of design scenarios is presented in Table 1.

Table 1. Scenarios considered in the FoS calculations

Scenario	Groundwater total head	Reservoir water depth	Base of reservoir level	Base of aquitard level
1A Reservoir EOC ¹ using EA ² data	21.5mAOD. Maximum historic EA data from the LL interpolated at the reservoir site.	0m (empty)	5mAOD	-40mAOD to -64mAOD
1B Reservoir EOC using the numerical model	23.6 mAOD Modelled maximum groundwater levels in the LL if there was no abstraction.	0m (empty)	5mAOD	-40mAOD to -64mAOD
1C Reservoir EOC using VWP data	The maximum recorded water pressure from each VWP installation.	0m (empty)	5mAOD	For aquifer data: top of aquifer unit For aquitard data: install level of the VWP
2A Reservoir operation using EA data	21.5mAOD. Maximum historic EA data from the LL interpolated at the reservoir site.	2.7m (low water level)	5mAOD	-40mAOD to -64mAOD

¹End of Construction ²Environment Agency

RESULTS AND DISCUSSION

End of Construction

The results plotted in Figure 4 show for the scenarios in which a constant groundwater total head (taken from the LL) is applied, the FoS against uplift in the LL and BWL between -40mAOD and -64mAOD (the continuous coloured lines) decreases with increased elevation (i.e. shallower depth). Over the same depth range the results based on monitored VWP data (the individual-coloured points), are broadly consistent with these lines with a FoS at around 1.5. At elevations of around -35 to -30 mAOD, in the CB and KLS, the FoS is slightly higher at 1.6 to 1.7. However, where the top of the KLS is at higher elevations (-23 to -18 mAOD), the FoS is at its lowest at 1.3 to 1.4. One shallower piezometer in the OXC gives a FoS against uplift of

Some considerations in assessing hydraulic uplift risk for large reservoirs on overconsolidated clays

around 1.5. In summary, for a reservoir base level of 5mAOD, no results sit above a FoS of 2.0 but all results sit above a FoS of 1.3.

These results provide an initial answer to the first of our key questions: the critical horizon for the calculation of uplift risk is theoretically the top of the Kellaways Sand stratum (KLS), and this is supported by some of the monitoring data. Where slightly higher FoS values are calculated, this may be due to deeper strata interface levels or lower recorded piezometric pressures in this complex, slightly 'leaky', multi-aquifer system.

In response to the third of the key questions, the spatial variation in FoS for the KLS and LL can be seen in Figure 5. Values are only reported for zones where monitoring points exist. At this stage a pattern emerges of the FoS for the KLS increasing from West to East. This assessment therefore opens the possibility of increasing excavation depth from West to East to maximise storage volume while maintaining safe conditions. However, no pattern is evident in the LL results and clearly more monitoring data is required.

In summary, the results highlight the need for site wide monitoring of porewater pressures in all strata given the potential variability in results. The results also show uplift risk should not be limited to a single stratum.

Reservoir Operation

As expected the operational case, line 2a in Figure 4, tracks to the right (higher FoS) of the EOC case (line 1a) given the minimum operational water level provides an additional stabilising force. In addition, for the operational case it is doubtful that unusually high-water pressures in the aquifers would coincide with a "1 in 500 yr drought", when presumably river levels and precipitation would be very low. These two assumptions seem incompatible, and it is unlikely such scenarios are realistic. Given these points, the end of construction case could be treated as temporary and a lower FoS accepted; the Contractor would be on site and appropriate monitoring could be used to implement an observational approach to the excavation depth vs aquifer pressures.

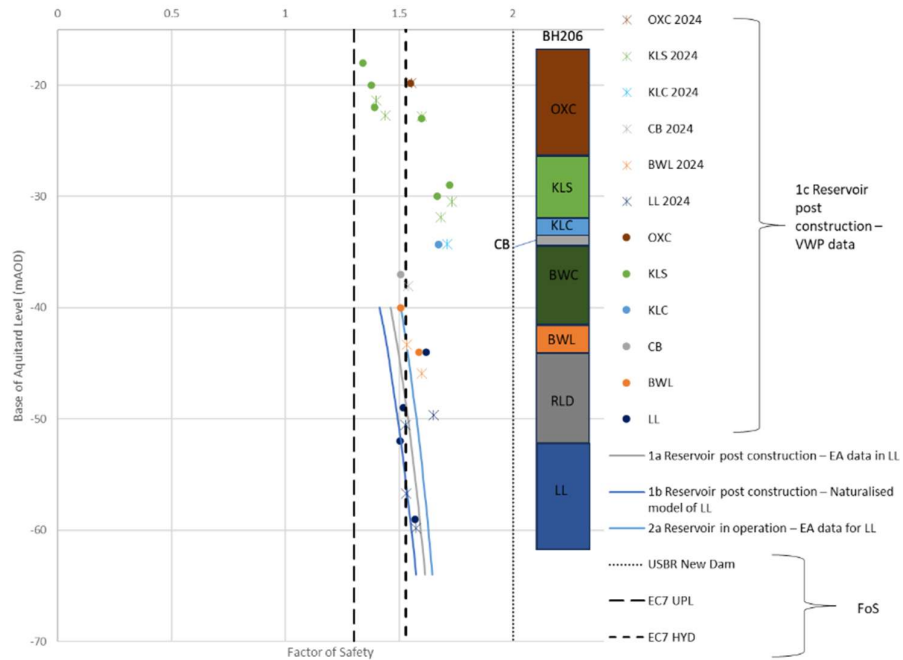


Figure 4. Summary of FoS against uplift for a borrow pit level of 5mAOD. A log of strata elevations at BH206 is included to show representative levels but the strata elevations change across the reservoir footprint.

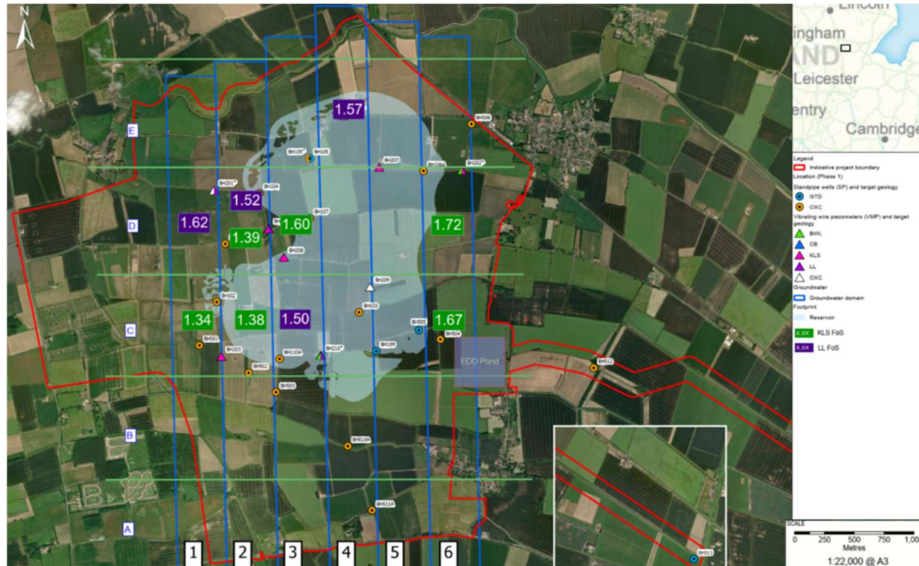


Figure 5. Map of the reservoir site with the FoS for KLS and LL for a borrow pit base at 5mAOD, labelled at grid locations where monitoring data currently is available. © Crown Copyright and database rights 2026, OS 100022432.

Some considerations in assessing hydraulic uplift risk for large reservoirs on overconsolidated clays

MITIGATIONS AND RISK MANAGEMENT

If the estimated FoS against uplift is judged insufficient at any time during construction, mitigation measures may be required to increase the FoS. These could take a variety of forms, as follows:

- Re-filling excavations in critical locations, using stockpiled excavated soils or abstracted water (see management of water levels below);
- Development of the borrow pit in “cells”, firstly to work at a scale that would allow side-friction forces to be taken into account for a particular cell, and secondly to allow easier “re-fill” in an emergency scenario, as discussed above;
- Management of the water pressure within the underlying aquifer, which could be achieved through the following methods:
 - ~ installing bespoke abstraction wells within the Lincolnshire Limestone, which will remove water from the highest-pressure aquifer which governs the pressure of the whole system, thereby relieving pressure from the aquifers above it by ‘under-drainage’;
 - ~ increasing the abstraction rate at the existing Anglian Water abstraction wells in the area, which in turn will reduce the pressure within the Lincolnshire Limestone;
 - ~ a combination of installation of new wells and increase in abstraction rate from the existing wells, possibly combined with the use of this water, temporarily, for ‘re-filling’ excavations as discussed above.

If any of the abstraction methods above are used, the development of a hydrogeological groundwater model would be required to simulate the effect of varying the abstraction rate within the LL and to estimate the resulting pressure to be used in a revised uplift assessment. Careful calibration of location and abstraction rate of the wells would be required to provide a satisfactory FoS against uplift in all areas. In addition, a key preparatory intervention is to discuss abstraction and discharge licensing with regulators in parallel with developing the proposed construction methods and programme.

Spatial variation of excavation level within the borrow pit may also be explored, with excavation level deeper in the areas with higher FoS against uplift and shallower where the estimated FoS is lower.

SUMMARY

This paper focuses on an early study undertaken to assess the uplift risk for the base of the Lincolnshire reservoir. The construction of the reservoir will involve the excavation of substantial volumes of clay bedrock to considerable depths and underlying artesian aquifers pose a risk of hydraulic failure due to uplift of the formation clays.

In developing the assessment methodology, a number of key questions came to light which are discussed within this paper, and summarised below;

1. Which horizon is the critical one for hydraulic uplift?

These results of the hydraulic uplift assessment provide an initial answer; the critical horizon for the calculation of uplift risk is theoretically the top of the Kellaways Sand stratum (KLS), and this is supported by some of the monitoring data. Where slightly higher FoS values are calculated, this may be due to deeper strata interface levels or lower recorded piezometric pressures in this complex, slightly 'leaky', multi-aquifer system.

2. What groundwater level should be taken for each design scenario?

Given the observed variability in groundwater pressures and the influence of reservoir operating conditions, it was not considered appropriate to adopt a single "representative" groundwater level for all design scenarios. While Eurocode 7 provides a general framework, it also acknowledges that for structures of the highest consequence class, additional or amended provisions may be required, introducing a degree of ambiguity in its application to dam-related uplift assessments. In this context, and consistent with the intent of considering unfavourable conditions, a range of plausible and adverse groundwater levels have been adopted to reflect the uncertainty in both measured and modelled data, with different values applied to represent the governing conditions for each design scenario. However, this approach should not be considered definitive, and there remains a need for further monitoring and further project-specific assessment as the design develops, a likely construction sequence begins to emerge and any regulatory constraints become clearer. In addition, the two available standards of assessment stand some distance apart. Therefore, clearer guidance or standardised approaches for addressing uplift mechanisms in high-consequence hydraulic structures would also be beneficial.

3. Is the risk of hydraulic uplift constant across the reservoir site?

The risk of hydraulic uplift is not constant across the reservoir site. This is a product of variation in stratigraphy owing to the regional dip of the bedrock

Some considerations in assessing hydraulic uplift risk for large reservoirs on overconsolidated clays

strata and spatial variation in modelled and observed groundwater levels. The KLS shows an increase in FOS from west to east, suggesting potential for deeper excavation in the eastern area. In contrast, no clear pattern is observed in the LL, highlighting the need for additional monitoring. Overall, the results demonstrate that uplift risk varies across both location and strata, requiring site-wide assessment rather than a focus on a single horizon.

This paper has highlighted the implications of uncertainty in groundwater conditions and uplift response within a complex, multi-aquifer system. It is recommended that groundwater levels for design are defined using a range of plausible and adverse conditions informed by observed and modelled data, supported by enhanced targeted monitoring to better constrain variability across the site. This will also be linked in with the construction sequence and a thorough understanding of any abstraction and discharge constraints. There is also a need for updated guidance to support a more consistent and robust approach to uplift assessment for high-consequence hydraulic structures.

ACKNOWLEDGMENTS

The authors would like to thank Anglian Water for permission to publish this paper, and to colleagues in Mott MacDonald for assistance and support.

REFERENCES

- British Geological Survey (1996), *1:50,000 Series, England and Wales Sheet 127, Grantham, Solid and Drift Geology*
- Moore J. F. A and Longworth. T. I (1978), Hydraulic uplift of the base of a deep excavation in Oxford Clay. *Geotechnique*, **29(1)** pp 35-46
- USBR (2014), *Design Standards No. 13: Embankment Dams: Chapter 8: Seepage*
- BSI (2013), *BS EN 1997-1:2004+A1:2013 Eurocode 7: Geotechnical design – general rules*
- BSI (2014), *NA+A1:2014 to BS EN 1997-1:2004+A1:2013 UK national annex to Eurocode 7*